PRELIMINARY GEOTECHNICAL INVESTIGATION
&
GEOLOGIC SEISMIC HAZARD REPORT

PROPOSED CAMPUS IMPROVEMENTS
HUNTINGTON PARK HIGH SCHOOL
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LOS ANGELES UNIFIED SCHOOL DISTRICT
LOS ANGELES, CALIFORNIA

PROJECT NO. A8326-06-62

JULY 14, 2015

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A8326-06-62
July 14, 2015

Mr. Peyman Soroosh Moghadam
Los Angeles Unified School District
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Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
AND GEOLOGIC SEISMIC HAZARD REPORT
PROPOSED CAMPUS IMPROVEMENTS
HUNTINGTON PARK HIGH SCHOOL
6020 MILES AVENUE, HUNTINGTON PARK, CALIFORNIA

Reference: *Geotechnical Investigation, Track and Field Upgrade, Huntington Park High School, 6020 Miles Avenue, Huntington Park, California, LAUSD Project 56.40036*, prepared by Geocon Inland Empire, Inc. December 16, 2008, Project No. A8326-06-33.

Dear Mr. Moghadam:

In accordance with your authorization of our proposal dated April 9, 2015, we have performed a preliminary geotechnical investigation and geologic seismic hazard report for the proposed campus improvements located at 6020 Miles Avenue in the City of Huntington Park, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our preliminary investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

The primary intent of this report is to address the potential geologic hazards and geotechnical conditions that could impact site development and to provide preliminary recommendations. Additional analyses will be required in order to provide comprehensive geotechnical recommendations for design and construction. This report is not intended to be submitted to DSA/CGS.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,
GEOCON WEST, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the proposed campus improvements located at the Huntington Park High School campus located at 6020 Miles Avenue in the City of Huntington Park, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction. Due to the preliminary nature of the project at this time, the primary intent of this report is to address the potential geologic hazards and geotechnical conditions that could impact site development and to provide preliminary recommendations. Additional borings and engineering analyses will be required in order to provide comprehensive geotechnical recommendations for design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on June 16, 2015, by excavating six 8-inch diameter borings to depths of approximately 20½ and 60½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located the Huntington Park High School campus located at 6020 Miles Avenue in the City of Huntington Park, California. The campus is an L-shaped parcel bounded by Slauson Avenue to the north, by Oak Street to the east, by Belgrave Avenue and Randolph Street to the south, and by Miles Avenue to the west. The site is relatively level, with no pronounced highs or lows. Surface water drainage appears to be by sheet flow along the ground surface to existing area drains and the city streets. Vegetation in the area of the site consists of grass, shrubs and trees.

Based on the information provided by the Client, it is our understanding that the proposed project is preliminary in nature and consists of several campus renovations. A new three-story on-grade classroom structure will be constructed in the southern portion of the site, subsequent to demolition of several existing buildings. Several other existing buildings will undergo varying level of modernization and seismic retrofitting, including the administration/classroom building, the home economics building, the annex building, the gymnasium building, and the welding shop. In addition, new paving may be constructed. Based on the preliminary nature of the project, it is our further understanding that the preliminary geotechnical report is not intended for submittal to DSA/CGS.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed classroom structure will be up to 300 kips, and wall loads will be up to 5 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

Based on our review of published geologic maps of the area, the site is located in the northern portion of the Los Angeles Basin. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier fault to the east, the Palos Verdes Peninsula and Pacific Ocean on the west, and the Santa Ana Mountains and San Joaquin Hills on the south. The Los Angeles Basin is northwest-trending alluviated lowland plain, sometimes called the Coastal Plain of Los Angeles. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits, which rest on a basement complex of presumably igneous and metamorphic composition (Yerkes, et al., 1965). The basement surface within the central portion of the basin extends to a maximum depth of 32,000 feet below sea level. The prominent structural features within the Los Angeles Basin include the central lowland plain, the uplifted Palos Verdes Hills, and the northwest trending line of low hills and mesas (underlain by the Newport-Inglewood fault zone). The site is shown with respect to local geologic features on Figure 3, Local Geologic Map.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill overlying Holocene Age alluvial sediments generally consisting of unconsolidated sand, silt, clay, and gravel. These units are underlain by several hundred feet of poorly consolidated sediments of the Pleistocene Age Lakewood Formation (California Department of Water Resources, 1961). The site is shown with respect to local geologic conditions on Figure 3, Local Geologic Map. Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 4 feet below existing ground surface. The artificial fill generally consists of brown silty sand. The artificial fill is characterized as slightly moist and medium-dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

The artificial fill materials are underlain by relatively flat-lying Holocene age alluvial basin deposits generally consisting of fine-grained to coarse grained sand with silt and silty sand. The soils are primarily slightly moist and medium and become denser with increased depth. The Holocene deposits extend to an approximate depth of 120 feet beneath the ground surface and are underlain at depth by upper Pleistocene age continental deposits of the Lakewood Formation.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the South Gate 7.5 Minute Quadrangle, Los Angeles County, California (California Division of Mines and Geology [CDMG], 1999), the historically highest groundwater level in the area is approximately 30 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

The Los Angeles County Department of Public Works (LADPW) has maintained various wells in the vicinity of the subject site over the past 50 years. The closest groundwater monitoring well to the site is Well No. 1491C located approximately 3,000 feet to the east (LADPW, 2015a). Review of the available monitoring data for this well for the monitoring period between 1946 and 2010 indicate that the depth to groundwater has fluctuated between 69 and 309 feet below the existing ground surface. The most recent groundwater level measurement for Well No. 1491C was measured in August, 2010 at a depth of 175 feet below the existing ground surface (LADPW, 2015a).

The site is located within the Central Basin of Los Angeles County (see figure on the following page). According to the State of California Department of Water Resources (DWR), groundwater development in the Central Basin increased dramatically in 1909, with the advent of the deep-well turbine pump. With time the demand for groundwater exceeded the natural replenishment of water within the Central Basin. This overdraft affected the groundwater situation in the basin by lowering the water levels and by causing oceanfront areas to be subject to sea water intrusion.

In 1950, the Central Basin Water Association was formed to address the deteriorating groundwater situation in the Central Basin, and to develop a water management plan. In 1959 the Central and West Basin Water Replenishment District (CWBWRD) was formed, with the objective to replenish and maintain the groundwater basin by purchasing imported water, recharging basins and halting sea water intrusion. In 1962, the CWBWRD brought litigation against 700 defendants, and sought to obtain title to the right to use groundwater and regulate withdrawals from the Central Basin to protect the water supply from deterioration. As a result, in 1962 the DWR was appointed as Watermaster to manage the groundwater within the Central Basin. The Watermaster's primary responsibility is to administer the water management plan and issue annual reports to the Court on groundwater related events within the Basin. As part of this plan, every groundwater pumper in the basin provides DWR a monthly report of its extractions.

The CWBWRD has since changed its name to the Water Replenishment District of Southern California (WRD). The WRD's hydrogeologists and engineers closely monitor, collect data and manage the groundwater resources of the District throughout the year, utilizing a computer model developed by the United States Geological Survey (USGS) to simulate groundwater conditions and to predict future conditions. The DWR cooperates closely with the WRD to maintain a balance between outflow/extractions and replenishment of groundwater by natural and artificial recharge. The 1962 Judgment limits the extraction of groundwater from the Basin to 217,367 acre-feet annually, so imported water has become a major component of the area's water supply. WRD tracks groundwater in the production wells and monitoring wells located throughout the District to observe the conditions of the basins and to identify any up or down trends that may impact groundwater resources. .

There is a high annual demand for groundwater within the Central Basin, and the Basin has an overdraft every year, which means that pumping exceeds natural groundwater replenishment. The overdraft is made up by purchasing artificial replenishment (imported and recycled water) to help make up the difference.

According to a 2002 report by DWR, historical groundwater extractions and the use of imported water within the Central Basin for the years 1957-58 to 2002-03 indicate that the amount of groundwater extracted has remained essentially the same since 1962-63 and the imported water usage has been consistent since 1983-84.

In conjunction with the WRD, the DWR systematically and continuously monitors and manages the groundwater use and changes in water levels within the Basin. Over the past 45 years, the withdrawal of groundwater by pumping has remained essentially constant, and every year there is an overdraft, because the amount pumped exceeds the amount of natural replenishment. The amount pumped also does not meet the total annual demand for water. As a result, the WRD purchases imported water and reclaimed water to meet the high demand as well as to maintain a balance between the water extracted and the natural replenishment that occurs. This approach does not significantly change the annual amount of water stored in the Basin. Going forward, if the management plan used for the past 45 years is maintained, there should not be any significant change in the depth to groundwater compared to the range of current levels. In fact, as the population increases the demand will certainly continue to increase.

According to the Groundwater Elevation Contours Map below (WRD, 2005) the depth of groundwater in the area of the site is at an approximate Elevation -32 feet MSL. The ground surface elevation of the site is about 175 feet MSL; therefore, the depth of groundwater is approximately 207 feet below the ground surface, which is relatively consistent with the water level measurements observed in Well No. 1491C.

Groundwater was not encountered in our field explorations, drilled to a maximum depth of 60½ feet below the existing ground surface. Based on the lack of groundwater in our borings, and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.18).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as California Division of Mines and Geology [CDMG]) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Avalon Compton strand of the Newport Inglewood Fault Zone located approximately 5.9 miles southwest of the site (Ziony and Jones, 1989). Other nearby active faults are the Raymond Fault, the Hollywood Fault, the Whittier Fault and the Verdugo Fault Zone located 9.0 miles north, 9.0 miles north, 10½ miles east and 11 miles north of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault zone is located approximately 37 miles northeast of the site.

The closest potentially active fault to the site is the Coyote Pass Fault located approximately 3.4 miles north of the site. Other nearby potentially active fault are the MacArthur Park Fault, the Norwalk Fault, and the Overland Fault located approximately 4.5 miles northwest, 9.0 miles southeast, and 9.3 miles west of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust. The site is located within the vertical projection of the Los Angeles segment of the Puente Hills Blind Thrust Fault. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	72	ESE
Near Redlands	July 23, 1923	6.3	55	E
Long Beach	March 10, 1933	6.4	29	SE
Tehachapi	July 21, 1952	7.5	83	NW
San Fernando	February 9, 1971	6.6	31	NNW
Whittier Narrows	October 1, 1987	5.9	10	ENE
Sierra Madre	June 28, 1991	5.8	23	ENE
Landers	June 28, 1992	7.3	103	E
Big Bear	June 28, 1992	6.4	81	E
Northridge	January 17, 1994	6.7	24	NW
Hector Mine	October 16, 1999	7.1	119	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes summarizes the seismic design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.010g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.706g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.010g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.059g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.340g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.706g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.745g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.745g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.64 magnitude event occurring at a hypocentral distance of 9.1 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.63 magnitude occurring at a hypocentral distance of 15.4 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

Liquefaction involves a sudden loss in strength of saturated, cohesionless soils that are subject to ground vibration and results in temporary transformation of the soil to a fluid mass. If the liquefying layer is near the surface, the effects are much like that of quicksand for any structure located on it. If the layer is deeper in the subsurface, it may provide a sliding surface for the material above it.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

A review of the State of California Seismic Hazard Zone Map for the South Gate Quadrangle (CDMG, 1999) indicates that the site is located in an area designated as "liquefiable" (see Figure 6). However, according to the Los Angeles County Safety Element (1990), the site is not located within an area identified as having a potential for liquefaction.

As indicated in the *Groundwater* section of this report (see Section 5.0) the depth of groundwater is in excess of 100 feet.

Without the presence of shallow groundwater the site soils would not be prone to liquefaction; however, the soils may be prone to settlement as a result of earthquake shaking as indicated in the following section.

6.5 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9.

The calculations provided herein for boring B6 indicate that the alluvial soils to a depth of 50 feet below the ground surface could be prone to approximately 0.25 inches of settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M), respectively.

The calculations provided herein for boring B6 indicate that the alluvial soils to a depth of 50 feet below the ground surface could be prone to approximately 0.51 inches of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M), respectively.

Calculation of the anticipated seismically-induced settlements is provided as Figures 7 through 10.

6.6 Slope Stability

The site and surrounding vicinity is relatively flat and not within an area identified as having a potential for slope instability. Also, the site is not within an area designated as having a potential for seismic slope instability (CDMG, 1998). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Hansen Dam and Sepulveda Dam inundation areas. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.8 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2008; Los Angeles County, 2015).

6.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is not located within the limits of an oilfield and oil wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the preliminary investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction. This report should be considered "preliminary" and the geotechnical design parameters presented herein should be reviewed and updated as the project progresses to a more finalized state.

7.1.2 Up to 4 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 The enclosed seismically induced settlement analyses indicate that the site soils could be prone to approximately ¼ inch of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). Differential settlement at the foundation level is anticipated to be less than ¼ inch over a distance of 30 feet.

7.1.4 Based on these considerations, the proposed classroom structure may be supported on a conventional foundation system deriving support in newly placed engineered fill. It is recommended that the upper 5 feet of existing earth materials in the building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of three feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.5 As an alternative, a reinforced concrete mat foundation system deriving support in newly placed engineered fill may also be utilized for support of the proposed structures. A mat foundation system is more capable of distributing loads and minimizing potential differential settlements.

7.1.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed classroom structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of adjacent improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.17).

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.10 Based on the granular nature of the soils encountered during site exploration, it is likely that a stormwater infiltration system is feasible for this project. Furthermore, based on our prior investigation conducted within the track and field portion of the campus (Geocon Project No. A8326-06-33), the upper ten feet of alluvial soils were found to be highly conductive to infiltration. The soils encountered during this investigation are similar to the soil encountered in the track and field area of the site. The determination of the soil percolation rate for the design of a stormwater infiltration system was beyond the scope of this investigation and, if required, can be addressed as an addendum under separate cover. It is recommended that percolation testing be performed once the project progresses to a more finalized plan so that infiltration rates can be determined at the proper depths and locations. Geocon can provide input on the recommended setback from the infiltration system to existing and proposed structures.

7.1.11 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

7.1.12 Any changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.2.4 The upper five feet of existing site soils encountered during this investigation are considered to have a “very low” expansive potential (EI = 0); and the soils are classified as “non-expansive” in accordance with the 2013 California Building Code (CBC) Section 1803.5.3 Recommendations presented herein assume that foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B9) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

7.4.1 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

7.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.4.3 Grading should commence with the removal of existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved in writing by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established, it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.4 As a minimum, it is recommended that the upper five feet of existing earth materials within the proposed building footprint area should be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove existing artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of three feet beyond the building footprint area or a distance equal to the depth of fill below the foundation, whichever is greater.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper twelve inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.4.7 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.17).

7.4.8 Prior to construction of exterior slabs and paving, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.10 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. Import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B9). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable. Prior to placing any bedding materials or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.12 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding sands, fill, steel, gravel, or concrete.

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 15 and 25 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Conventional Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed structures provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of three feet of newly placed engineered fill.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.4 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing value of 4,000 psf.

7.6.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.6.6 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.

7.6.7 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as-needed to maintain the required three-foot-thick blanket of engineered fill below new foundations.

7.6.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.6.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

7.6.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.6.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.7 Mat Foundation Design

7.7.1 As an alternative, a reinforced concrete mat foundation system deriving support in newly placed engineered fill may be utilized for support of the proposed structures. A mat foundation system is more capable of distributing loads and minimizing potential differential settlements. Foundations should be underlain by a minimum of three feet of newly placed engineered fill.

7.7.2 The recommended maximum allowable bearing value is 4,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in newly placed engineered fill. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

7.7.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

7.7.5 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between concrete slab and new placed engineered fill without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.7.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.7.7 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Foundation Settlement

7.8.1 The enclosed seismically induced settlement analysis indicate that the site soils could be prone to approximately ¼ inch of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The differential settlement at the foundation level is anticipated to be less than ¼ inch over a distance of 30 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.8.2 The maximum expected settlement for a conventional foundation system with a maximum allowable bearing value of 4,000 psf and deriving support in the recommended bearing material is estimated to be approximately 1 inch and occur below the heaviest loaded structural element. Differential settlement is expected to be less than ½ inch over a distance of 20 feet.

7.8.3 The maximum settlement for a reinforced concrete mat foundation designed with the maximum allowable bearing value of 4,000 psf and deriving support in the recommended bearing materials is expected to be less than ¾ inches and occur below the heaviest loaded structural element. Differential settlement between the center and corner of the mat is not expected to exceed ½ inch.

7.8.4 Based on seismic considerations, the proposed structure should be designed for a combined static and seismically induced differential settlement of ¾ inches over a distance of 20 feet.

7.8.5 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines or existing structures, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.9.2 If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.40 may be used with the dead load forces in properly compacted engineered fill and undisturbed alluvial soils.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 260 pounds per cubic foot (pcf) with a maximum earth pressure of 2,600 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).

7.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete

Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.4 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.11.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

7.11.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

7.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.13 Retaining Walls Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report (see Section 7.6).

7.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.13.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.13.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

7.13.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.13.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 11). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 12).

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design and Retaining Wall Design* section of this report (see Sections 7.6 and 7.13).

7.15.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses.

7.15.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.14).

7.15.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.16 Elevator Piston

7.16.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation, especially if the drilling is performed subsequent to the foundation construction.

7.16.2 Casing may be required if caving is experienced in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.17 Temporary Excavations

7.17.1 Excavations up to 5 feet in height may be required during grading and construction operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than five feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.17.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special temporary excavation measures can be provided under separate cover, if needed.

7.17.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 Surface Drainage

7.18.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.18.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.18.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.18.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.19 Plan Review

7.19.1 Grading and foundation plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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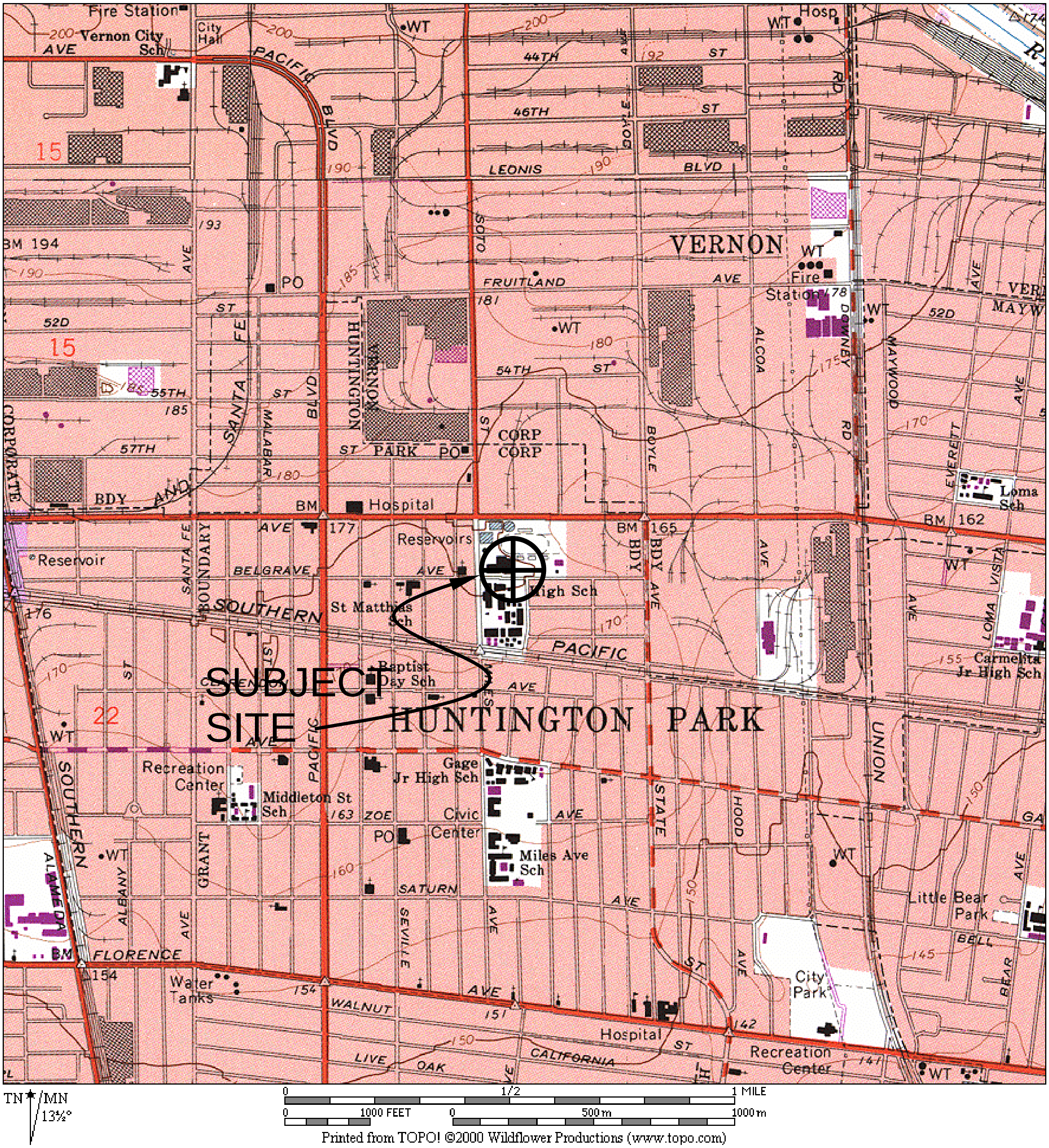

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, SOUTH GATE, CA QUADRANGLE

LATITUDE: 33.9862
LONGTITUDE: -118.2188

GEOCON WEST, INC.

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PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RMA | CHECKED BY: SFK

VICINITY MAP

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015 | PROJECT NO. A8326-06-62 | FIG. 1

LEGEND

B6 Approximate Location of Boring

Approximate Limits of Subject Site

Proposed Seismic Evaluation/Retrofit

Demolish and Construct 3-Story Classroom Building

0 150' 300'

GEOCON W E S T, I N C.

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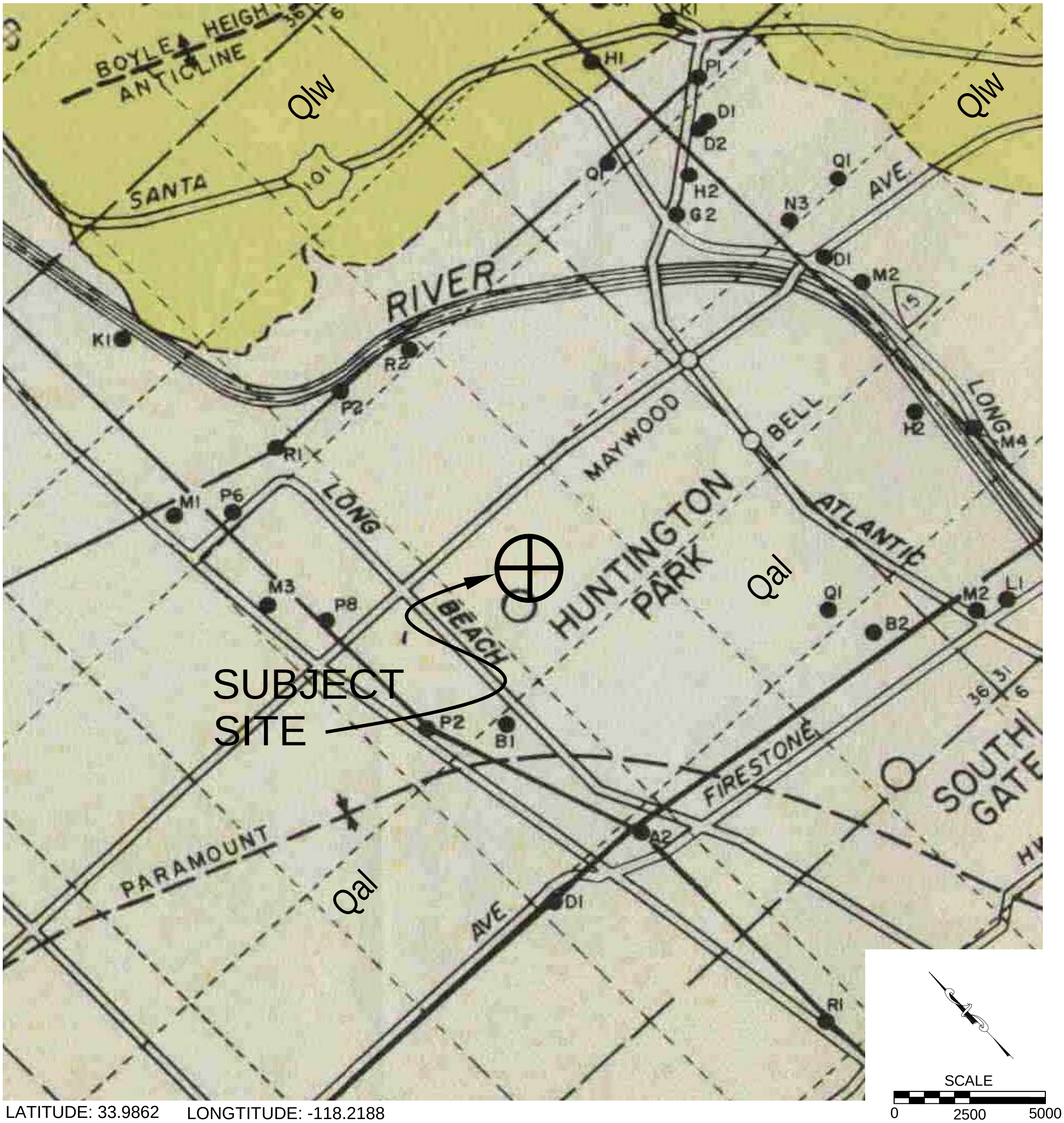
PZ		9000

SITE PLAN

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. 2



LATITUDE: 33.9862 LONGTITUDE: -118.2188

Qal - YOUNGER ALLUVIUM: poorly consolidated gravel, sand, silt and clay

Qlw - LAKEWOOD FORMATION: Marine and continental gravel, sand, sandy silt, silt and clay with shale pebbles

REFERENCE: State of California Department of Water Resources, Southern California Region ,1961, Ground Water Geology of the Coastal Plain of Los Angeles County

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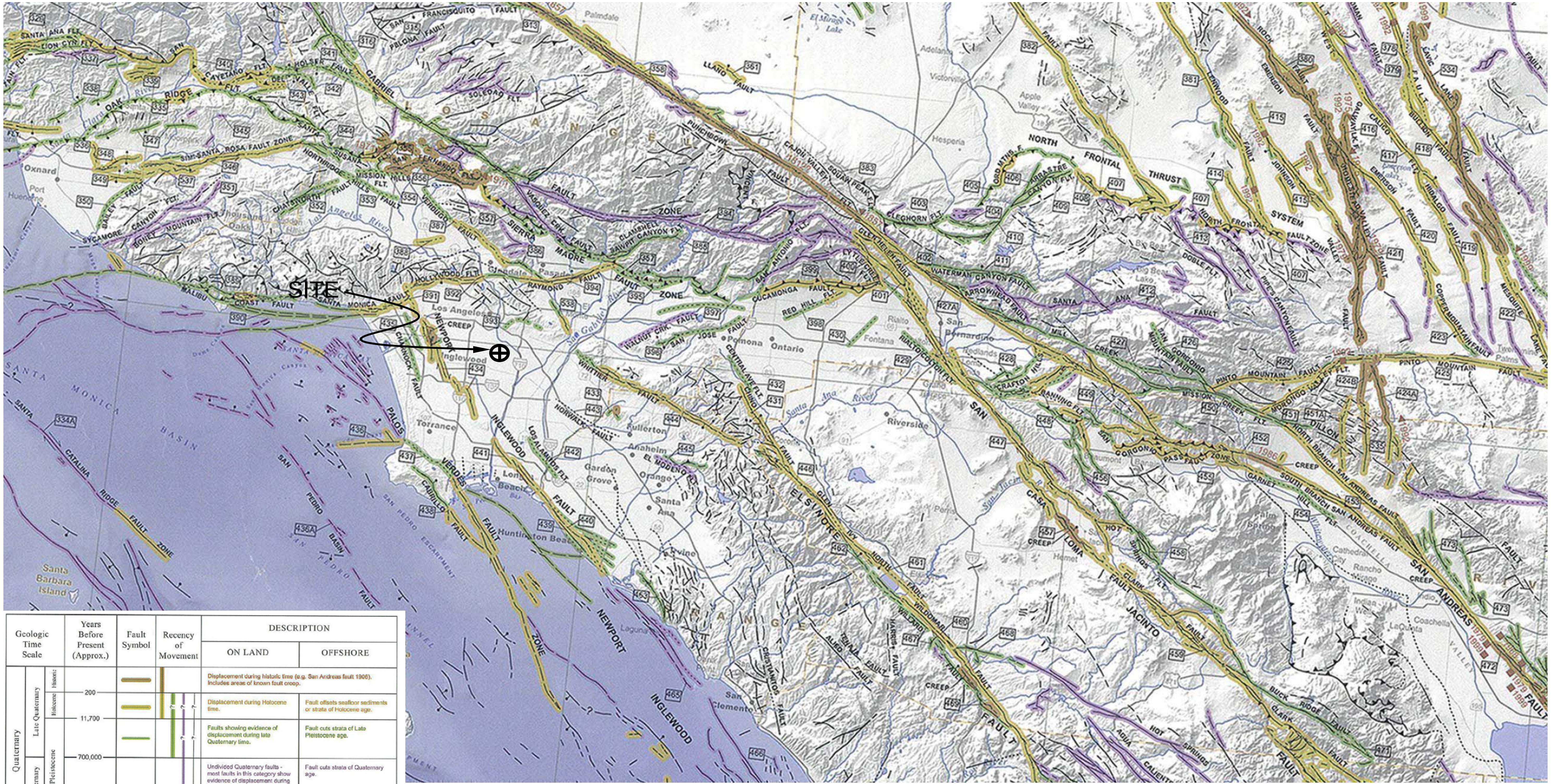
DRAFTED BY: RMA | CHECKED BY: SFK

LOCAL GEOLOGIC MAP

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015 | PROJECT NO. A8326-06-62 | FIG. 3

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene			? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene			? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary				?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000*				
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

0 12 24 Miles

LATITUDE: 33.9862
LONGTITUDE: -118.2188

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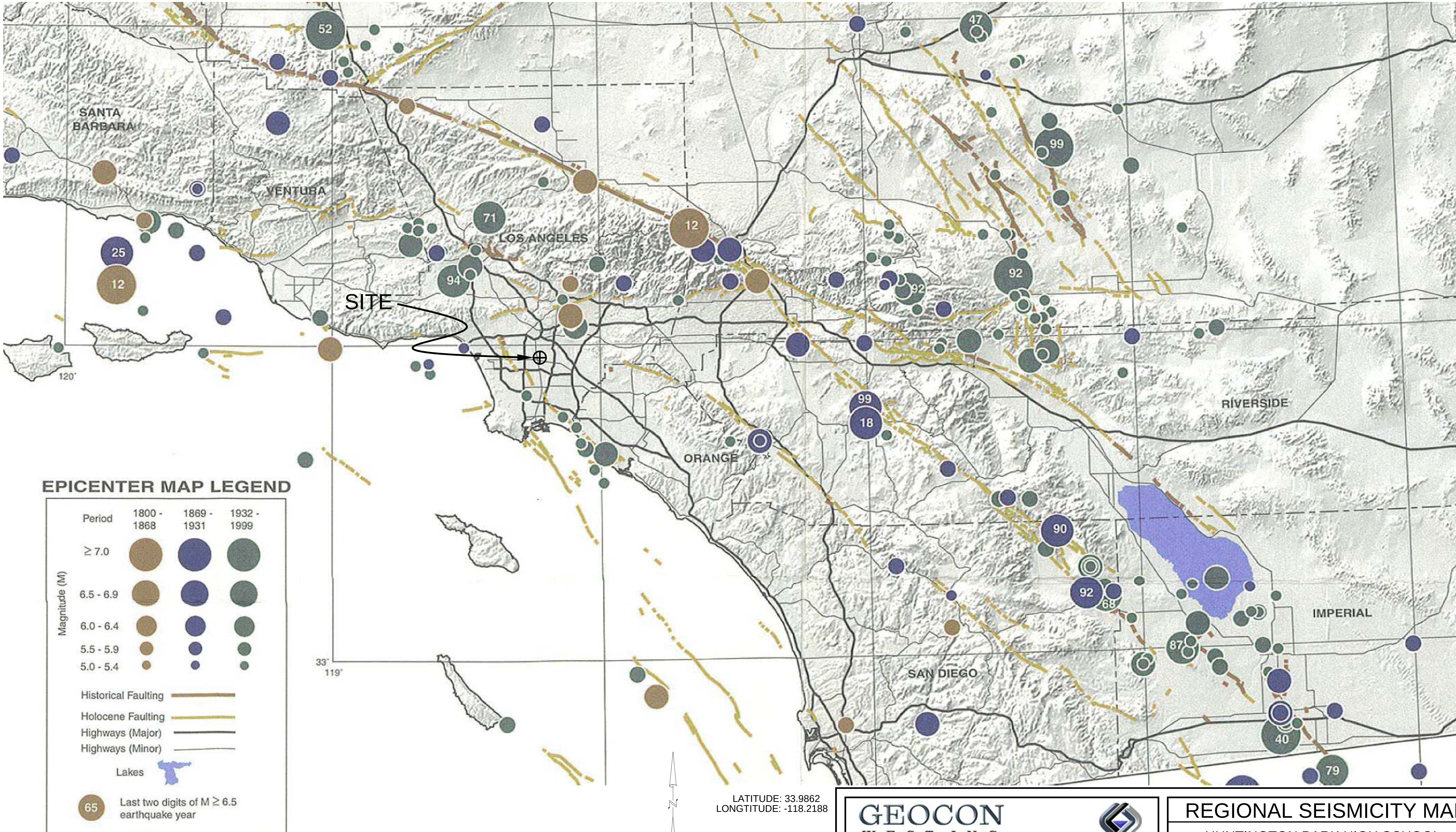
DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL FAULT MAP

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015 | PROJECT NO. A8326-06-62 | FIG. 4



LATITUDE: 33.9862
LONGTITUDE: -118.2188

Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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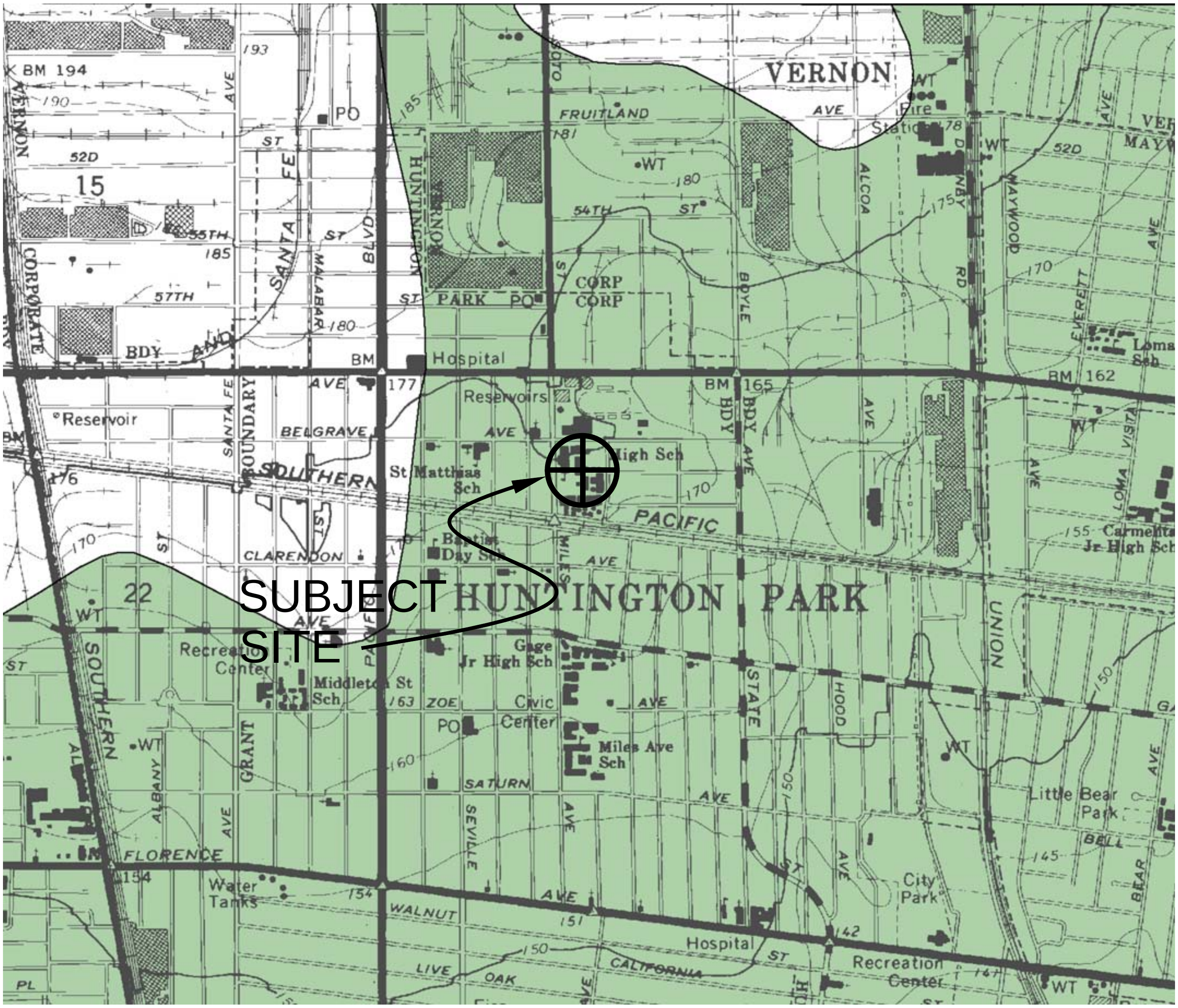
DRAFTED BY: RMA	CHECKED BY: SFK

REGIONAL SEISMICITY MAP

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. 5



LATITUDE: 33.9862
LONGTITUDE: -118.2188

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

REFERENCE: C.D.M.G. SEISMIC HAZARD ZONE MAP, SOUTH GATE QUADRANGLE, CA

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DRAFTED BY: RMA | CHECKED BY: SFK

SEISMIC HAZARD ZONE MAP

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015 | PROJECT NO. A8326-06-62 | FIG. 6

GEOCON

Project Name: LAUSD Huntington Park
Project No: A8326-06-62
Boring 6

DESIGN EARTHQUAKE - EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
Peak Horiz. Acceleration PGA_M (g):	0.745
2/3 PGA_M (g):	0.497
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	100.0
Groundwater Depth During Exploration	100.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes)	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	141.5	0	18.0	2.5	1	0	96	2.000	46.6	141.5	Infin.	0.998	0.236	--
2.0	141.5	0	18.0	2.5	1	0	96	2.000	46.6	141.5	Infin.	0.993	0.235	--
3.0	141.5	0	18.0	2.5	1	0	96	2.000	46.6	141.5	Infin.	0.989	0.234	--
4.0	141.5	0	18.0	2.5	1	0	96	2.000	46.6	141.5	Infin.	0.984	0.233	--
5.0	104.5	0	11.0	5.0	1	11	71	1.838	27.5	104.5	0.336	0.979	0.232	--
6.0	104.5	0	11.0	5.0	1	11	71	1.700	25.5	104.5	0.295	0.975	0.231	--
7.0	104.5	0	11.0	5.0	1	11	71	1.589	24.0	104.5	0.269	0.970	0.230	--
8.0	104.5	0	11.0	5.0	1	11	71	1.497	22.7	104.5	0.251	0.966	0.229	--
9.0	127.7	0	3.0	10.0	1	44	34	1.412	12.5	127.7	0.136	0.961	0.228	--
10.0	127.7	0	3.0	10.0	1	44	34	1.333	12.2	127.7	0.133	0.957	0.226	--
11.0	127.7	0	3.0	10.0	1	44	34	1.266	11.9	127.7	0.130	0.952	0.225	--
12.0	127.7	0	3.0	10.0	1	44	34	1.208	11.7	127.7	0.128	0.947	0.224	--
13.0	127.7	0	3.0	10.0	1	44	34	1.157	11.5	127.7	0.126	0.943	0.223	--
14.0	127.7	0	10.0	15.0	1	44	56	1.113	22.5	127.7	0.248	0.938	0.222	--
15.0	127.7	0	10.0	15.0	1	44	56	1.073	21.9	127.7	0.241	0.934	0.221	--
16.5	127.7	0	10.0	15.0	1	44	56	1.029	21.3	127.7	0.233	0.928	0.220	--
17.0	106.9	0	16.0	20.0	1	4	66	1.014	25.0	106.9	0.281	0.923	0.219	--
18.0	106.9	0	16.0	20.0	1	4	66	0.982	24.3	106.9	0.269	0.920	0.218	--
19.0	106.9	0	16.0	20.0	1	4	66	0.959	23.7	106.9	0.260	0.915	0.217	--
20.0	106.9	0	16.0	20.0	1	4	66	0.937	23.1	106.9	0.253	0.911	0.216	--
21.5	106.9	0	16.0	20.0	1	4	66	0.912	22.5	106.9	0.245	0.905	0.214	--
22.0	125.5	0	16.0	25.0	1	6	62	0.901	24.0	125.5	0.257	0.901	0.213	--
23.0	125.5	0	16.0	25.0	1	6	62	0.875	23.3	125.5	0.248	0.897	0.212	--
24.0	125.5	0	16.0	25.0	1	6	62	0.855	22.8	125.5	0.241	0.893	0.211	--
25.0	125.5	0	16.0	25.0	1	6	62	0.837	22.4	125.5	0.235	0.888	0.210	--
26.5	125.5	0	16.0	25.0	1	6	62	0.816	21.8	125.5	0.228	0.882	0.209	--
27.0	115.5	0	25.0	27.5	1	6	75	0.808	34.4	115.5	Infin.	0.878	0.208	--
28.0	115.5	0	25.0	27.5	1	6	75	0.791	33.7	115.5	Infin.	0.874	0.207	--
29.0	115.5	0	25.0	27.5	1	6	75	0.777	33.1	115.5	Infin.	0.870	0.206	--
30.5	115.5	0	13.0	30.0	1	27	53	0.762	22.3	115.5	0.226	0.864	0.205	--
31.0	107.8	0	16.0	35.0	1	8	56	0.756	21.5	107.8	0.211	0.859	0.203	--
32.0	107.8	0	16.0	35.0	1	8	56	0.742	21.1	107.8	0.207	0.856	0.203	--
33.0	107.8	0	16.0	35.0	1	8	56	0.732	20.9	107.8	0.204	0.851	0.202	--
34.0	107.8	0	16.0	35.0	1	8	56	0.722	20.6	107.8	0.201	0.847	0.200	--
35.0	107.8	0	16.0	35.0	1	8	56	0.712	20.3	107.8	0.198	0.842	0.199	--
36.0	107.8	0	16.0	35.0	1	8	56	0.703	20.1	107.8	0.196	0.838	0.198	--
37.0	110.3	0	25.0	40.0	1	8	67	0.694	30.6	110.3	Infin.	0.833	0.197	--
38.0	110.3	0	25.0	40.0	1	8	67	0.686	30.2	110.3	Infin.	0.829	0.196	--
39.0	110.3	0	25.0	40.0	1	8	67	0.677	29.9	110.3	0.404	0.824	0.195	--
40.0	110.3	0	25.0	40.0	1	3	67	0.669	28.9	110.3	0.331	0.819	0.194	--
41.0	110.3	0	25.0	40.0	1	3	67	0.661	28.5	110.3	0.319	0.815	0.193	--
42.0	111.7	0	25.0	40.0	1	3	67	0.654	28.2	111.7	0.310	0.810	0.192	--
43.0	111.7	0	25.0	40.0	1	3	67	0.647	27.9	111.7	0.301	0.806	0.191	--
44.0	111.7	0	25.0	45.0	1	3	65	0.639	27.6	111.7	0.286	0.801	0.190	--
45.0	111.7	0	25.0	45.0	1	4	65	0.633	27.3	111.7	0.280	0.797	0.189	--
46.0	111.7	0	25.0	45.0	1	4	65	0.626	27.0	111.7	0.274	0.792	0.187	--
47.0	110.6	0	25.0	45.0	1	4	65	0.619	26.7	110.6	0.269	0.787	0.186	--
48.0	110.6	0	25.0	45.0	1	4	65	0.613	26.4	110.6	0.265	0.783	0.185	--
49.0	110.6	0	21.0	50.0	1	52	57	0.607	29.0	110.6	0.319	0.778	0.184	--
50.0	110.6	0	21.0	50.0	1	52	57	0.601	28.8	110.6	0.311	0.774	0.183	--

GEOCON

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

DESIGN EARTHQUAKE

DE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
Peak Horiz. Acceleration (g):	0.497

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Fig 4.1 Relative Density [Dr] (%)	Fig 4.2 Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[γeff]*[Geff] / [Gmax]	Fig 4.4 γeff Shear Strain	[γeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	141.5	0.04	0.02	0.011	18	1.25	96.1	2.0	46.6	1.0	247.581	4.57E-05	6.00E-05	0.006	2.18E-03	8.1527	1.65E-03	0.00
2.0	1.0	1.5	141.5	0.11	0.07	0.034	18	1.25	96.1	2.0	46.6	1.0	428.822	7.76E-05	1.40E-04	0.014	5.08E-03	8.1527	3.86E-03	0.00
3.0	1.0	2.5	141.5	0.18	0.12	0.057	18	1.25	96.1	2.0	46.6	1.0	553.607	9.82E-05	1.60E-04	0.016	5.80E-03	8.1527	4.41E-03	0.00
4.0	1.0	3.5	141.5	0.25	0.17	0.080	18	1.25	96.1	2.0	46.6	1.0	655.036	1.14E-04	1.70E-04	0.017	6.16E-03	8.1527	4.69E-03	0.00
5.0	1.0	4.5	104.5	0.24	0.16	0.076	11	1.25	71.2	1.8	27.5	1.0	535.667	1.30E-04	1.70E-04	0.017	1.16E-02	8.1527	8.81E-03	0.00
6.0	1.0	5.5	104.5	0.29	0.19	0.093	11	1.25	71.2	1.7	25.5	1.0	577.748	1.44E-04	1.70E-04	0.017	1.27E-02	8.1527	9.63E-03	0.00
7.0	1.0	6.5	104.5	0.34	0.23	0.109	11	1.25	71.2	1.6	24.0	1.0	614.852	1.57E-04	1.50E-04	0.015	1.21E-02	8.1527	9.18E-03	0.00
8.0	1.0	7.5	104.5	0.39	0.26	0.126	11	1.25	71.2	1.5	22.7	1.0	648.234	1.69E-04	1.50E-04	0.015	1.29E-02	8.1527	9.82E-03	0.00
9.0	1.0	8.5	127.7	0.54	0.36	0.174	3	1.25	33.8	1.4	12.5	1.0	625.135	2.38E-04	4.50E-04	0.045	7.93E-02	8.1527	6.02E-02	0.01
10.0	1.0	9.5	127.7	0.61	0.41	0.194	3	1.25	33.8	1.3	12.2	1.0	655.437	2.49E-04	4.50E-04	0.045	8.17E-02	8.1527	6.21E-02	0.01
11.0	1.0	10.5	127.7	0.67	0.45	0.214	3	1.25	33.8	1.3	11.9	1.0	684.132	2.59E-04	4.50E-04	0.045	8.38E-02	8.1527	6.37E-02	0.02
12.0	1.0	11.5	127.7	0.73	0.49	0.234	3	1.25	33.8	1.2	11.7	0.9	711.447	2.68E-04	4.50E-04	0.045	8.57E-02	8.1527	6.52E-02	0.02
13.0	1.0	12.5	127.7	0.80	0.53	0.254	3	1.25	33.8	1.2	11.5	0.9	737.563	2.76E-04	3.70E-04	0.037	7.19E-02	8.1527	5.47E-02	0.01
14.0	1.0	13.5	127.7	0.86	0.58	0.273	10	1.25	56.3	1.1	22.5	0.9	958.629	2.25E-04	3.70E-04	0.037	3.22E-02	8.1527	2.44E-02	0.01
15.0	1.0	14.5	127.7	0.93	0.62	0.293	10	1.25	56.3	1.1	21.9	0.9	985.260	2.31E-04	3.70E-04	0.037	3.31E-02	8.1527	2.52E-02	0.01
16.5	1.5	15.8	127.7	1.01	0.67	0.317	10	1.25	56.3	1.0	21.3	0.9	1017.139	2.38E-04	3.70E-04	0.037	3.43E-02	8.1527	2.61E-02	0.01
17.0	0.5	16.8	106.9	0.90	0.60	0.281	16	1.25	66.4	1.0	25.0	0.9	1012.671	2.09E-04	3.70E-04	0.037	2.83E-02	8.1527	2.15E-02	0.00
18.0	1.0	17.5	106.9	0.94	0.63	0.293	16	1.25	66.4	1.0	24.3	0.9	1024.169	2.14E-04	3.70E-04	0.037	2.94E-02	8.1527	2.23E-02	0.01
19.0	1.0	18.5	106.9	0.99	0.66	0.309	16	1.25	66.4	1.0	23.7	0.9	1044.605	2.18E-04	3.70E-04	0.037	3.02E-02	8.1527	2.30E-02	0.01
20.0	1.0	19.5	106.9	1.04	0.70	0.324	16	1.25	66.4	0.9	23.1	0.9	1064.283	2.21E-04	3.70E-04	0.037	3.11E-02	8.1527	2.36E-02	0.01
21.5	1.5	20.8	106.9	1.11	0.74	0.343	16	1.25	66.4	0.9	22.5	0.9	1087.916	2.26E-04	3.70E-04	0.037	3.21E-02	8.1527	2.44E-02	0.01
22.0	0.5	21.8	125.5	1.36	0.91	0.421	16	1.25	62.3	0.9	24.0	0.9	1233.687	2.41E-04	3.70E-04	0.037	2.97E-02	8.1527	2.26E-02	0.00
23.0	1.0	22.5	125.5	1.41	0.95	0.434	16	1.25	62.3	0.9	23.3	0.9	1242.597	2.45E-04	3.70E-04	0.037	3.07E-02	8.1527	2.34E-02	0.01
24.0	1.0	23.5	125.5	1.47	0.99	0.451	16	1.25	62.3	0.9	22.8	0.9	1260.541	2.48E-04	3.70E-04	0.037	3.16E-02	8.1527	2.40E-02	0.01
25.0	1.0	24.5	125.5	1.54	1.03	0.468	16	1.25	62.3	0.8	22.4	0.9	1277.995	2.51E-04	3.00E-04	0.030	2.63E-02	8.1527	2.00E-02	0.00
26.5	1.5	25.8	125.5	1.62	1.08	0.489	16	1.25	62.3	0.8	21.8	0.9	1299.173	2.54E-04	3.00E-04	0.030	2.71E-02	8.1527	2.06E-02	0.01
27.0	0.5	26.8	115.5	1.54	1.03	0.465	25	1.25	75.2	0.8	34.4	0.9	1478.820	2.10E-04	3.00E-04	0.030	1.56E-02	8.1527	1.19E-02	0.00
28.0	1.0	27.5	115.5	1.59	1.06	0.477	25	1.25	75.2	0.8	33.7	0.9	1488.502	2.12E-04	3.00E-04	0.030	1.61E-02	8.1527	1.22E-02	0.00
29.0	1.0	28.5	115.5	1.65	1.10	0.491	25	1.25	75.2	0.8	33.1	0.9	1506.839	2.14E-04	3.00E-04	0.030	1.64E-02	8.1527	1.25E-02	0.00
30.5	1.5	29.8	115.5	1.72	1.15	0.510	13	1.25	53.0	0.8	22.3	0.9	1349.714	2.44E-04	3.00E-04	0.030	2.63E-02	8.1527	2.00E-02	0.01
31.0	0.5	30.8	107.8	1.66	1.11	0.489	16	1.25	56.2	0.8	21.5	0.9	1310.290	2.39E-04	3.00E-04	0.030	2.75E-02	8.1527	2.09E-02	0.00
32.0	1.0	31.5	107.8	1.70	1.14	0.499	16	1.25	56.2	0.7	21.1	0.9	1318.438	2.41E-04	3.00E-04	0.030	2.81E-02	8.1527	2.13E-02	0.01
33.0	1.0	32.5	107.8	1.75	1.17	0.512	16	1.25	56.2	0.7	20.9	0.9	1333.150	2.42E-04	3.00E-04	0.030	2.85E-02	8.1527	2.17E-02	0.01
34.0	1.0	33.5	107.8	1.81	1.21	0.525	16	1.25	56.2	0.7	20.6	0.8	1347.557	2.43E-04	3.00E-04	0.030	2.90E-02	8.1527	2.20E-02	0.01
35.0	1.0	34.5	107.8	1.86	1.25	0.537	16	1.25	56.2	0.7	20.3	0.8	1361.674	2.44E-04	3.00E-04	0.030	2.94E-02	8.1527	2.24E-02	0.01
36.0	1.0	35.5	107.8	1.91	1.28	0.549	16	1.25	56.2	0.7	20.1	0.8	1375.516	2.45E-04	3.00E-04	0.030	2.99E-02	8.1527	2.27E-02	0.01
37.0	1.0	36.5	110.3	2.01	1.35	0.574	25	1.25	67.3	0.7	30.6	0.8	1623.464	2.15E-04	3.00E-04	0.030	1.80E-02	8.1527	1.37E-02	0.00
38.0	1.0	37.5	110.3	2.07	1.39	0.586	25	1.25	67.3	0.7	30.2	0.8	1638.819	2.16E-04	3.00E-04	0.030	1.83E-02	8.1527	1.39E-02	0.00
39.0	1.0	38.5	110.3	2.12	1.42	0.597	25	1.25	67.3	0.7	29.9	0.8	1653.900	2.16E-04	3.00E-04	0.030	1.85E-02	8.1527	1.41E-02	0.00
40.0	1.0	39.5	110.3	2.18	1.46	0.609	25	1.25	67.3	0.7	28.9	0.8	1656.312	2.18E-04	3.00E-04	0.030	1.93E-02	8.1527	1.47E-02	0.00
41.0	1.0	40.5	110.3	2.23	1.50	0.620	25	1.25	67.3	0.7	28.5	0.8	1670.625	2.19E-04	3.00E-04	0.030	1.96E-02	8.1527	1.49E-02	0.00
42.0	1.0	41.5	111.7	2.32	1.55	0.639	25	1.25	67.3	0.7	28.2	0.8	1695.710	2.21E-04	3.00E-04	0.030	1.99E-02	8.1527	1.51E-02	0.00
43.0	1.0	42.5	111.7	2.37	1.59	0.650	25	1.25	67.3	0.6	27.9	0.8	1709.561	2.21E-04	3.00E-04	0.030	2.01E-02	8.1527	1.53E-02	0.00
44.0	1.0	43.5	111.7	2.43	1.63	0.660	25	1.25	64.7	0.6	27.6	0.8	1723.192	2.21E-04	3.00E-04	0.030	2.04E-02	8.1527	1.55E-02	0.00
45.0	1.0	44.5	111.7	2.49	1.67	0.670	25	1.25	64.7	0.6	27.3	0.8	1736.610	2.21E-04	3.00E-04	0.030	2.07E-02	8.1527	1.57E-02	0.00
46.0	1.0	45.5	111.7	2.54	1.70	0.680	25	1.25	64.7	0.6	27.0	0.8	1749.826	2.22E-04	3.00E-04	0.030	2.09E-02	8.1527	1.59E-02	0.00
47.0	1.0	46.5	110.6	2.57	1.72	0.683	25	1.25	64.7	0.6	26.7	0.8	1753.674	2.21E-04	3.00E-04	0.030	2.12E-02	8.1527	1.61E-02	0.00
48.0	1.0	47.5	110.6	2.63	1.76	0.692	25	1.25	64.7	0.6	26.4	0.8	1766.498	2.21E-04	3.00E-04	0.030	2.15E-02	8.1527	1.63E-02	0.00
49.0	1.0	48.5	110.6	2.68	1.80	0.701	21	1.25	57.1	0.6	29.0	0.8	1840.629	2.13E-04	3.00E-04	0.030	1.92E-02	8.1527	1.46E-02	0.00
50.0	1.0	49.5	110.6	2.74	1.83	0.710	21	1.25	57.1	0.6	28.8	0.8	1854.972	2.13E-04	3.00E-04	0.030	1.94E-02	8.1527	1.47E-02	0.00

TOTAL SETTLEMENT = **0.25**

Figure 8

GEOCON

Project Name: LAUSD Huntington Park
Project No: A8326-06-62
Boring 6

MAXIMUM CONSIDERED EARTHQUAKE - EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD
By Thomas F. Blake (1994-1996)
LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
Peak Horiz. Acceleration PGA_M (g):	0.745
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	100.0
Groundwater Depth During Exploration	100.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes)	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	141.5	0	18.0	2.5	1		96	2.000	46.6	141.5	Infin.	0.998	0.355	--
2.0	141.5	0	18.0	2.5	1		96	2.000	46.6	141.5	Infin.	0.993	0.354	--
3.0	141.5	0	18.0	2.5	1		96	2.000	46.6	141.5	Infin.	0.989	0.352	--
4.0	141.5	0	18.0	2.5	1		96	2.000	46.6	141.5	Infin.	0.984	0.351	--
5.0	104.5	0	11.0	5.0	1	11	71	1.838	27.5	104.5	0.336	0.979	0.349	--
6.0	104.5	0	11.0	5.0	1	11	71	1.700	25.5	104.5	0.295	0.975	0.347	--
7.0	104.5	0	11.0	5.0	1	11	71	1.589	24.0	104.5	0.269	0.970	0.346	--
8.0	104.5	0	11.0	5.0	1	11	71	1.497	22.7	104.5	0.251	0.966	0.344	--
9.0	127.7	0	3.0	10.0	1	44	34	1.412	12.5	127.7	0.136	0.961	0.342	--
10.0	127.7	0	3.0	10.0	1	44	34	1.333	12.2	127.7	0.133	0.957	0.341	--
11.0	127.7	0	3.0	10.0	1	44	34	1.266	11.9	127.7	0.130	0.952	0.339	--
12.0	127.7	0	3.0	10.0	1	44	34	1.208	11.7	127.7	0.128	0.947	0.338	--
13.0	127.7	0	3.0	10.0	1	44	34	1.157	11.5	127.7	0.126	0.943	0.336	--
14.0	127.7	0	10.0	15.0	1	44	56	1.113	22.5	127.7	0.248	0.938	0.334	--
15.0	127.7	0	10.0	15.0	1	44	56	1.073	21.9	127.7	0.241	0.934	0.333	--
16.5	127.7	0	10.0	15.0	1	44	56	1.029	21.3	127.7	0.233	0.928	0.331	--
17.0	106.9	0	16.0	20.0	1	4	66	1.014	25.0	106.9	0.281	0.923	0.329	--
18.0	106.9	0	16.0	20.0	1	4	66	0.982	24.3	106.9	0.269	0.920	0.328	--
19.0	106.9	0	16.0	20.0	1	4	66	0.959	23.7	106.9	0.260	0.915	0.326	--
20.0	106.9	0	16.0	20.0	1	4	66	0.937	23.1	106.9	0.253	0.911	0.325	--
21.5	106.9	0	16.0	20.0	1	4	66	0.912	22.5	106.9	0.245	0.905	0.322	--
22.0	125.5	0	16.0	25.0	1	6	62	0.901	24.0	125.5	0.257	0.901	0.321	--
23.0	125.5	0	16.0	25.0	1	6	62	0.875	23.3	125.5	0.248	0.897	0.320	--
24.0	125.5	0	16.0	25.0	1	6	62	0.855	22.8	125.5	0.241	0.893	0.318	--
25.0	125.5	0	16.0	25.0	1	6	62	0.837	22.4	125.5	0.235	0.888	0.316	--
26.5	125.5	0	16.0	25.0	1	6	62	0.816	21.8	125.5	0.228	0.882	0.314	--
27.0	115.5	0	25.0	27.5	1	6	75	0.808	34.4	115.5	Infin.	0.878	0.313	--
28.0	115.5	0	25.0	27.5	1	6	75	0.791	33.7	115.5	Infin.	0.874	0.311	--
29.0	115.5	0	25.0	27.5	1	6	75	0.777	33.1	115.5	Infin.	0.870	0.310	--
30.5	115.5	0	13.0	30.0	1	27	53	0.762	22.3	115.5	0.226	0.864	0.308	--
31.0	107.8	0	16.0	35.0	1	8	56	0.756	21.5	107.8	0.211	0.859	0.306	--
32.0	107.8	0	16.0	35.0	1	8	56	0.742	21.1	107.8	0.207	0.856	0.305	--
33.0	107.8	0	16.0	35.0	1	8	56	0.732	20.9	107.8	0.204	0.851	0.303	--
34.0	107.8	0	16.0	35.0	1	8	56	0.722	20.6	107.8	0.201	0.847	0.302	--
35.0	107.8	0	16.0	35.0	1	8	56	0.712	20.3	107.8	0.198	0.842	0.300	--
36.0	107.8	0	16.0	35.0	1	8	56	0.703	20.1	107.8	0.196	0.838	0.298	--
37.0	110.3	0	25.0	40.0	1	8	67	0.694	30.6	110.3	Infin.	0.833	0.297	--
38.0	110.3	0	25.0	40.0	1	8	67	0.686	30.2	110.3	Infin.	0.829	0.295	--
39.0	110.3	0	25.0	40.0	1	8	67	0.677	29.9	110.3	0.404	0.824	0.294	--
40.0	110.3	0	25.0	40.0	1	3	67	0.669	28.9	110.3	0.331	0.819	0.292	--
41.0	110.3	0	25.0	40.0	1	3	67	0.661	28.5	110.3	0.319	0.815	0.290	--
42.0	111.7	0	25.0	40.0	1	3	67	0.654	28.2	111.7	0.310	0.810	0.289	--
43.0	111.7	0	25.0	40.0	1	3	67	0.647	27.9	111.7	0.301	0.806	0.287	--
44.0	111.7	0	25.0	45.0	1	3	65	0.639	27.6	111.7	0.286	0.801	0.285	--
45.0	111.7	0	25.0	45.0	1	4	65	0.633	27.3	111.7	0.280	0.797	0.284	--
46.0	111.7	0	25.0	45.0	1	4	65	0.626	27.0	111.7	0.274	0.792	0.282	--
47.0	110.6	0	25.0	45.0	1	4	65	0.619	26.7	110.6	0.269	0.787	0.281	--
48.0	110.6	0	25.0	45.0	1	4	65	0.613	26.4	110.6	0.265	0.783	0.279	--
49.0	110.6	0	21.0	50.0	1	52	57	0.607	29.0	110.6	0.319	0.778	0.277	--
50.0	110.6	0	21.0	50.0	1	52	57	0.601	28.8	110.6	0.311	0.774	0.276	--

GEOCON

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

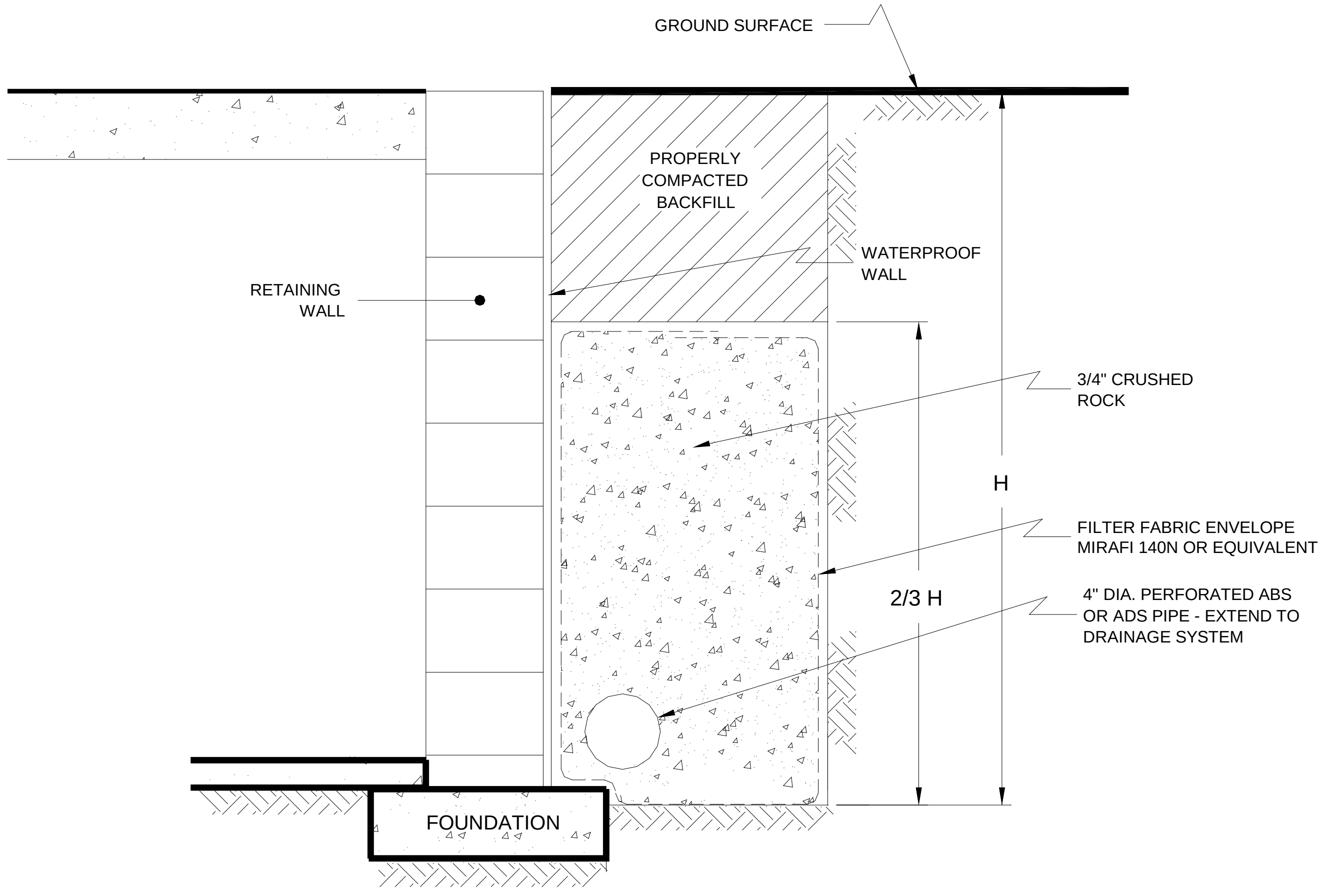
Earthquake Magnitude:	6.64
Peak Horiz. Acceleration (g):	0.745

Fig 4.1 Fig 4.2 Fig 4.4

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] / [Gmax]	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	141.5	0.04	0.02	0.017	18	1.25	96.1	2.0	46.6	1.0	247.581	6.85E-05	1.00E-04	0.010	3.63E-03	8.2202	2.77E-03	0.00
2.0	1.0	1.5	141.5	0.11	0.07	0.051	18	1.25	96.1	2.0	46.6	1.0	428.822	1.16E-04	2.30E-04	0.023	8.34E-03	8.2202	6.36E-03	0.00
3.0	1.0	2.5	141.5	0.18	0.12	0.086	18	1.25	96.1	2.0	46.6	1.0	553.607	1.47E-04	1.70E-04	0.017	6.16E-03	8.2202	4.70E-03	0.00
4.0	1.0	3.5	141.5	0.25	0.17	0.120	18	1.25	96.1	2.0	46.6	1.0	655.036	1.71E-04	1.70E-04	0.017	6.16E-03	8.2202	4.70E-03	0.00
5.0	1.0	4.5	104.5	0.31	0.21	0.149	11	1.25	71.2	1.8	27.5	1.0	614.061	2.23E-04	4.50E-04	0.045	3.07E-02	8.2202	2.34E-02	0.00
6.0	1.0	5.5	104.5	0.36	0.24	0.174	11	1.25	71.2	1.7	25.5	1.0	647.749	2.43E-04	4.50E-04	0.045	3.36E-02	8.2202	2.56E-02	0.01
7.0	1.0	6.5	104.5	0.41	0.28	0.199	11	1.25	71.2	1.6	24.0	1.0	678.420	2.60E-04	4.50E-04	0.045	3.62E-02	8.2202	2.76E-02	0.01
8.0	1.0	7.5	104.5	0.47	0.31	0.224	11	1.25	71.2	1.5	22.7	1.0	706.685	2.76E-04	4.50E-04	0.045	3.87E-02	8.2202	2.96E-02	0.01
9.0	1.0	8.5	127.7	0.52	0.35	0.252	3	1.25	33.8	1.4	12.5	1.0	614.304	3.51E-04	1.00E-03	0.100	1.76E-01	8.2202	1.34E-01	0.03
10.0	1.0	9.5	127.7	0.59	0.39	0.282	3	1.25	33.8	1.3	12.2	1.0	645.287	3.68E-04	1.00E-03	0.100	1.81E-01	8.2202	1.38E-01	0.03
11.0	1.0	10.5	127.7	0.65	0.44	0.312	3	1.25	33.8	1.3	11.9	1.0	674.553	3.83E-04	1.00E-03	0.100	1.86E-01	8.2202	1.42E-01	0.03
12.0	1.0	11.5	127.7	0.72	0.48	0.342	3	1.25	33.8	1.2	11.7	0.9	702.358	3.97E-04	1.00E-03	0.100	1.91E-01	8.2202	1.45E-01	0.03
13.0	1.0	12.5	127.7	0.78	0.52	0.372	3	1.25	33.8	1.2	11.5	0.9	728.898	4.09E-04	1.20E-03	0.120	2.33E-01	8.2202	1.78E-01	0.04
14.0	1.0	13.5	127.7	0.84	0.56	0.401	10	1.25	56.3	1.1	22.5	0.9	948.206	3.34E-04	7.10E-04	0.071	6.17E-02	8.2202	4.71E-02	0.01
15.0	1.0	14.5	127.7	0.91	0.61	0.430	10	1.25	56.3	1.1	21.9	0.9	975.289	3.43E-04	7.10E-04	0.071	6.36E-02	8.2202	4.85E-02	0.01
16.5	1.5	15.8	127.7	0.99	0.66	0.466	10	1.25	56.3	1.0	21.3	0.9	1007.666	3.54E-04	7.10E-04	0.071	6.58E-02	8.2202	5.02E-02	0.02
17.0	0.5	16.8	106.9	1.05	0.70	0.494	16	1.25	66.4	1.0	25.0	0.9	1095.787	3.40E-04	7.10E-04	0.071	5.42E-02	8.2202	4.14E-02	0.00
18.0	1.0	17.5	106.9	1.09	0.73	0.511	16	1.25	66.4	1.0	24.3	0.9	1104.757	3.45E-04	7.10E-04	0.071	5.63E-02	8.2202	4.30E-02	0.01
19.0	1.0	18.5	106.9	1.14	0.76	0.534	16	1.25	66.4	1.0	23.7	0.9	1122.512	3.51E-04	7.10E-04	0.071	5.80E-02	8.2202	4.42E-02	0.01
20.0	1.0	19.5	106.9	1.19	0.80	0.557	16	1.25	66.4	0.9	23.1	0.9	1139.722	3.56E-04	7.10E-04	0.071	5.96E-02	8.2202	4.55E-02	0.01
21.5	1.5	20.8	106.9	1.26	0.85	0.586	16	1.25	66.4	0.9	22.5	0.9	1160.529	3.61E-04	7.10E-04	0.071	6.16E-02	8.2202	4.70E-02	0.02
22.0	0.5	21.8	125.5	1.32	0.88	0.609	16	1.25	62.3	0.9	24.0	0.9	1212.091	3.55E-04	7.10E-04	0.071	5.69E-02	8.2202	4.34E-02	0.01
23.0	1.0	22.5	125.5	1.36	0.91	0.629	16	1.25	62.3	0.9	23.3	0.9	1221.577	3.60E-04	7.10E-04	0.071	5.90E-02	8.2202	4.50E-02	0.01
24.0	1.0	23.5	125.5	1.43	0.96	0.655	16	1.25	62.3	0.9	22.8	0.9	1240.131	3.65E-04	7.10E-04	0.071	6.06E-02	8.2202	4.62E-02	0.01
25.0	1.0	24.5	125.5	1.49	1.00	0.681	16	1.25	62.3	0.8	22.4	0.9	1258.154	3.70E-04	7.10E-04	0.071	6.21E-02	8.2202	4.74E-02	0.01
26.5	1.5	25.8	125.5	1.57	1.05	0.712	16	1.25	62.3	0.8	21.8	0.9	1279.990	3.75E-04	5.20E-04	0.052	4.69E-02	8.2202	3.58E-02	0.01
27.0	0.5	26.8	115.5	1.63	1.09	0.737	25	1.25	75.2	0.8	34.4	0.9	1519.355	3.23E-04	5.20E-04	0.052	2.71E-02	8.2202	2.07E-02	0.00
28.0	1.0	27.5	115.5	1.67	1.12	0.753	25	1.25	75.2	0.8	33.7	0.9	1528.204	3.26E-04	5.20E-04	0.052	2.78E-02	8.2202	2.12E-02	0.01
29.0	1.0	28.5	115.5	1.73	1.16	0.775	25	1.25	75.2	0.8	33.1	0.9	1545.638	3.28E-04	5.20E-04	0.052	2.84E-02	8.2202	2.17E-02	0.01
30.5	1.5	29.8	115.5	1.80	1.21	0.802	13	1.25	53.0	0.8	22.3	0.9	1383.025	3.75E-04	5.20E-04	0.052	4.56E-02	8.2202	3.48E-02	0.01
31.0	0.5	30.8	107.8	1.86	1.25	0.823	16	1.25	56.2	0.8	21.5	0.9	1387.772	3.80E-04	5.20E-04	0.052	4.76E-02	8.2202	3.63E-02	0.00
32.0	1.0	31.5	107.8	1.90	1.27	0.837	16	1.25	56.2	0.7	21.1	0.9	1394.596	3.82E-04	5.20E-04	0.052	4.87E-02	8.2202	3.71E-02	0.01
33.0	1.0	32.5	107.8	1.95	1.31	0.856	16	1.25	56.2	0.7	20.9	0.9	1407.851	3.83E-04	5.20E-04	0.052	4.95E-02	8.2202	3.77E-02	0.01
34.0	1.0	33.5	107.8	2.01	1.35	0.874	16	1.25	56.2	0.7	20.6	0.8	1420.870	3.84E-04	5.20E-04	0.052	5.03E-02	8.2202	3.83E-02	0.01
35.0	1.0	34.5	107.8	2.06	1.38	0.892	16	1.25	56.2	0.7	20.3	0.8	1433.661	3.85E-04	5.20E-04	0.052	5.10E-02	8.2202	3.89E-02	0.01
36.0	1.0	35.5	107.8	2.12	1.42	0.910	16	1.25	56.2	0.7	20.1	0.8	1446.237	3.86E-04	5.20E-04	0.052	5.18E-02	8.2202	3.95E-02	0.01
37.0	1.0	36.5	110.3	2.17	1.45	0.928	25	1.25	67.3	0.7	30.6	0.8	1685.998	3.35E-04	5.20E-04	0.052	3.12E-02	8.2202	2.38E-02	0.01
38.0	1.0	37.5	110.3	2.23	1.49	0.945	25	1.25	67.3	0.7	30.2	0.8	1700.292	3.35E-04	5.20E-04	0.052	3.17E-02	8.2202	2.42E-02	0.01
39.0	1.0	38.5	110.3	2.28	1.53	0.962	25	1.25	67.3	0.7	29.9	0.8	1714.355	3.36E-04	5.20E-04	0.052	3.21E-02	8.2202	2.45E-02	0.01
40.0	1.0	39.5	110.3	2.34	1.57	0.979	25	1.25	67.3	0.7	28.9	0.8	1715.349	3.39E-04	5.20E-04	0.052	3.35E-02	8.2202	2.55E-02	0.01
41.0	1.0	40.5	110.3	2.39	1.60	0.995	25	1.25	67.3	0.7	28.5	0.8	1728.727	3.39E-04	5.20E-04	0.052	3.40E-02	8.2202	2.59E-02	0.01
42.0	1.0	41.5	111.7	2.45	1.64	1.011	25	1.25	67.3	0.7	28.2	0.8	1741.986	3.40E-04	5.20E-04	0.052	3.44E-02	8.2202	2.63E-02	0.01
43.0	1.0	42.5	111.7	2.50	1.68	1.026	25	1.25	67.3	0.6	27.9	0.8	1755.131	3.40E-04	5.20E-04	0.052	3.49E-02	8.2202	2.66E-02	0.01
44.0	1.0	43.5	111.7	2.56	1.71	1.042	25	1.25	64.7	0.6	27.6	0.8	1768.083	3.40E-04	5.20E-04	0.052	3.54E-02	8.2202	2.70E-02	0.01
45.0	1.0	44.5	111.7	2.61	1.75	1.057	25	1.25	64.7	0.6	27.3	0.8	1780.847	3.40E-04	5.20E-04	0.052	3.58E-02	8.2202	2.73E-02	0.01
46.0	1.0	45.5	111.7	2.67	1.79	1.071	25	1.25	64.7	0.6	27.0	0.8	1793.431	3.41E-04	5.20E-04	0.052	3.63E-02	8.2202	2.77E-02	0.01
47.0	1.0	46.5	110.6	2.73	1.83	1.085	25	1.25	64.7	0.6	26.7	0.8	1805.777	3.41E-04	5.20E-04	0.052	3.67E-02	8.2202	2.80E-02	0.01
48.0	1.0	47.5	110.6	2.78	1.86	1.099	25	1.25	64.7	0.6	26.4	0.8	1817.893	3.41E-04	5.20E-04	0.052	3.72E-02	8.2202	2.84E-02	0.01
49.0	1.0	48.5	110.6	2.84	1.90	1.112	21	1.25	57.1	0.6	29.0	0.8	1893.092	3.29E-04	5.20E-04	0.052	3.33E-02	8.2202	2.54E-02	0.01
50.0	1.0	49.5	110.6	2.89	1.94	1.125	21	1.25	57.1	0.6	28.8	0.8	1906.790	3.29E-04	5.20E-04	0.052	3.36E-02	8.2202	2.56E-02	0.01

TOTAL SETTLEMENT = **0.51**

Figure 10



NO SCALE

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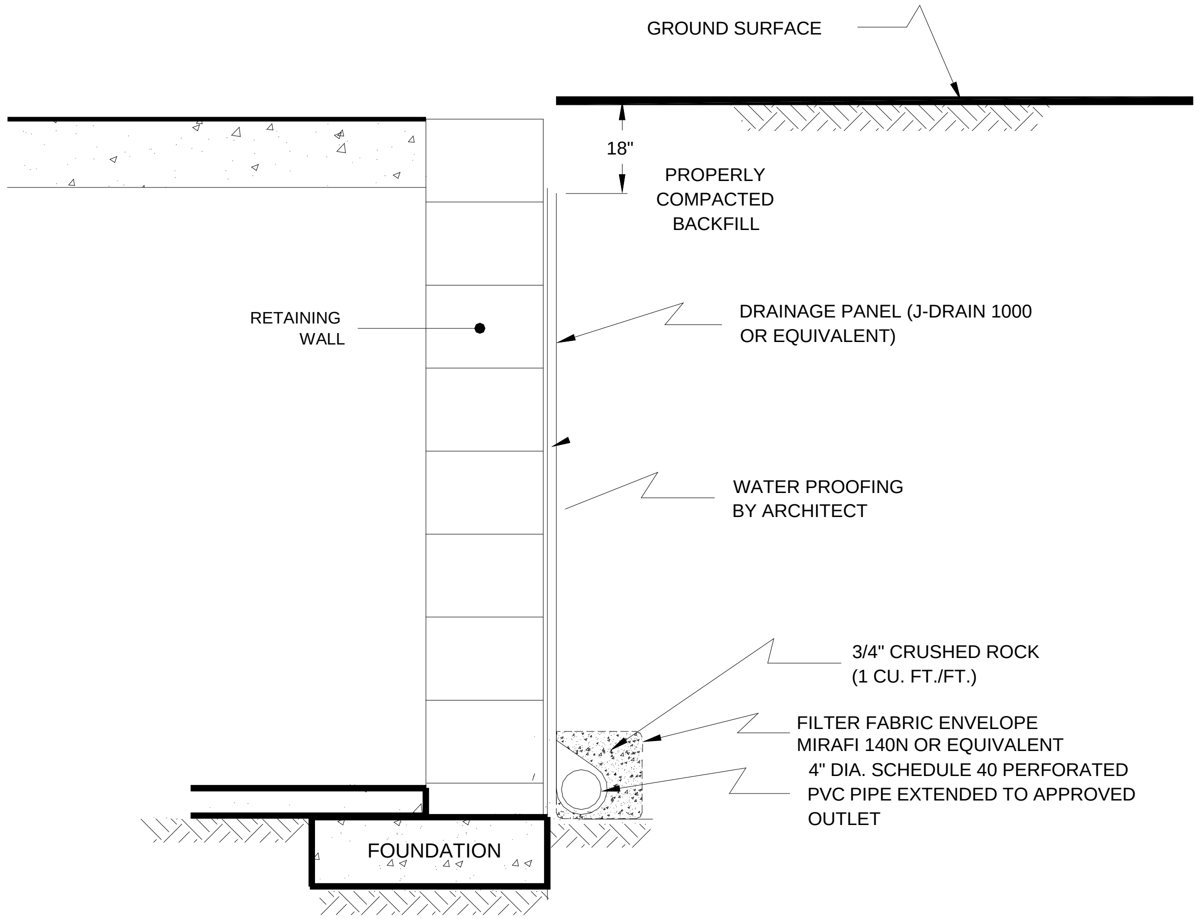
RETAINING WALL DRAIN DETAIL

HUNTINGTON PARK HIGH SCHOOL

LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. 11



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HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. 12

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on June 16, 2015 by excavating six 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 20½ and 60½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Standard Penetration Tests (SPTs) were performed in boring B6. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 4" BASE: 1.5"**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, brown, fine-grained.			
2					**ALLUVIUM** Silty Sand, very loose, slightly moist, grayish brown, fine-grained.			
	B1@3'			SM		6	100.3	7.8
4								
				SP-SM	Sand with Silt, loose, slightly moist, pale brown, fine-grained.			
6	B1@6'					13	100.0	1.5
				SM	Silty Sand, loose, slightly moist, grayish brown, fine-grained.			
8								
	B1@9'					16	109.0	8.3
10								
12	B1@12'				- medium dense	23	111.5	4.8
14								
	B1@15'					21	110.5	6.1
16								
18								
					- increase in silt content			
20	B1@20'					19	102.4	11.9

Total depth of boring: 20.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

A8326-06-62 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 1

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 6.5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown, fine-grained.			
2					**ALLUVIUM** Sand with Silt, very loose, slightly moist, pale brown, fine-grained, some medium-grained, trace coarse-grained.			
	B2@3'					5	92.3	4.6
4				SP-SM				
6	B2@6'				- increase in medium-grained	16	106.6	4.7
8					Silty Sand, loose, slightly moist, grayish brown, fine-grained.			
	B2@9'					7	90.6	17.6
10					- increase in silt content			
					- medium dense			
12	B2@12'					21	114.8	6.8
14				SM				
	B2@15'					22	113.6	6.3
16								
18								
					- increase in silt content			
20	B2@20'					26	107.5	14.1

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

Figure A2,
Log of Boring 2, Page 1 of 1

A8326-06-62 BORING LOGS.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING 3

ELEV. (MSL.) **- -** DATE COMPLETED **6/16/15**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		**DIRT**			
			ARTIFICIAL FILL			
2			Silty Sand, medium dense, slightly moist, dark brown, fine-grained, some medium-grained, trace fine gravel.			
	B3@3'			38	94.1	9.4
4			- some coarse-grained, some brick fragments			
			ALLUVIUM			
			Sand with Silt, loose, slightly moist, pale brown, fine- to coarse-grained.			
6	B3@6'	SP-SM	- fine- to medium-grained	13	96.7	4.3
8			- fine- to coarse-grained			
	B3@9'			11	92.3	4.3
10		SM	Silty Sand, loose, slightly moist, grayish brown, very fine- to fine-grained.			
12	B3@12'		- increase in silt content	13	92.1	7.8
14			- decrease in silt content, fine-grained			
	B3@15'			19	111.6	10.4
16						
18		SP-SM	Sand with Silt, medium dense, slightly moist, pale brown, fine-grained.			
20	B3@20'			44	106.3	11.5

Total depth of boring: 20.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface restored.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

A8326-06-62 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8326-06-62

BORING 4

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown, fine-grained.			
2				SM	**ALLUVIUM** Silty Sand, loose, slightly moist, grayish brown, fine-grained, trace medium- to coarse-grained.			
	B4@3'					8	106.2	3.3
4								
6	B4@6'			SP-SM	Sand with Silt, loose, slightly moist, pale brown, fine- to medium-grained, some coarse-grained.	10	100.5	1.3
8					- grayish brown			
	B4@9'					16	104.1	3.0
10				SM	Silty Sand, loose, slightly moist, grayish brown, fine-grained, some medium-grained.			
12	B4@12'					16	106.8	6.5
14								
	B4@15'					30	112.4	5.0
16								
18				SP-SM	Sand with Silt, medium dense, slightly moist, pale brown, fine- to coarse-grained.			
20	B4@20'					33	107.4	1.8

Total depth of boring: 20.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

A8326-06-62 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
☒ ... DISTURBED OR BAG SAMPLE
◧ ... STANDARD PENETRATION TEST
◪ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8326-06-62

BORING 5

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRASS**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, brown, fine-grained.			
2					**ALLUVIUM** Silty Sand, loose, slightly moist, brown, fine- to medium-grained.			
	B5@3'					5	101.0	3.5
4								
					- grayish brown			
6	B5@6'					9	97.9	7.2
8				SM	- decrease in silt content, some coarse-grained			
	B5@9'					9	109.1	4.5
10								
12					- medium dense, increase in silt content			
	B5@12'					22	113.1	6.2
14								
	B5@15'					17	107.4	6.5
16								
18				SP-SM	Sand with Silt, medium dense, slightly moist, pale brown, fine- to coarse-grained.			
20	B5@20'					46	107.4	6.5

Total depth of boring: 20.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface restored.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

Figure A5,
Log of Boring 5, Page 1 of 1

A8326-06-62 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8326-06-62

BORING 6

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3.5" BASE: 3"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained, some coarse-grained, some fine gravel, pods of yellowish brown silty sand.			
2	B6@2.5'					34	129.8	9.0
4	B6@5'			SP-SM	**ALLUVIUM** Sand with Silt, medium dense, slightly moist, grayish brown, fine- to coarse-grained.	11	--	4.3
6					- loose, pale brown, fine- to medium-grained			
8	B6@7.5'					8	103.3	1.2
				SM	Silty Sand, very loose, slightly moist, grayish brown, fine-grained.			
10	B6@10'					3	--	9.6
12	B6@12.5'				- medium dense, trace coarse-grained	24	114.4	11.6
14	B6@15'				- decrease in silt content, loose, pale brown	10	--	6.2
16				SP-SM	Sand with Silt, medium dense, slightly moist, pale brown, fine-grained.			
18	B6@17.5'					37	105.5	1.3
20	B6@20'				- some medium- to coarse-grained	16	--	2.3
22	B6@22.5'				- grayish brown, fine- to coarse-grained	19	114.2	9.9
24	B6@25'				- decrease in silt content	16	--	9.1
26					- pale brown, fine-grained, some medium-grained			
28	B6@27.5'					46	107.7	7.2
					Silty Sand, medium dense, slightly moist, grayish brown, fine- to			

A8326-06-62 BORING LOGS.GPJ

Figure A6,
Log of Boring 6, Page 1 of 3

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING 6

ELEV. (MSL.) - - DATE COMPLETED 6/16/15

EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B6@30'			SM	coarse-grained.	13	--	9.6
				SP-SM	Sand with Silt, medium dense, slightly moist, pale brown, fine- to medium-grained, trace coarse-grained.			
32	B6@32.5'					36	104.6	3.1
34	B6@35'					16	--	3.5
36					- dense, increase in medium- to coarse-grained			
38	B6@37.5'					55	107.8	2.3
40	B6@40'				- medium dense	25	--	2.5
42					- very dense, increase in coarse-grained, some fine gravel			
	B6@42.5'					50 (6")	108.9	2.6
44	B6@45'				- medium dense, fine- to medium-grained	25	--	2.9
46					- fine-grained			
48	B6@47.5'					43	108.4	2.0
				ML	Sandy Silt, stiff, slightly moist, grayish brown, fine-grained.			
50	B6@50'					21	--	14.5
52								
54					- increase in silt content, very fine-grained			
56	B6@55'					18	--	17.6
58								

Figure A6,
Log of Boring 6, Page 2 of 3

A8326-06-62 BORING LOGS.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 6** ELEV. (MSL.) **- -** DATE COMPLETED **6/16/15** EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B6@60'			ML	- decrease in silt content	20	--	13.0
					Total depth of boring: 60.5 feet Fill to 4 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.			

Figure A6,
Log of Boring 6, Page 3 of 3

A8326-06-62 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

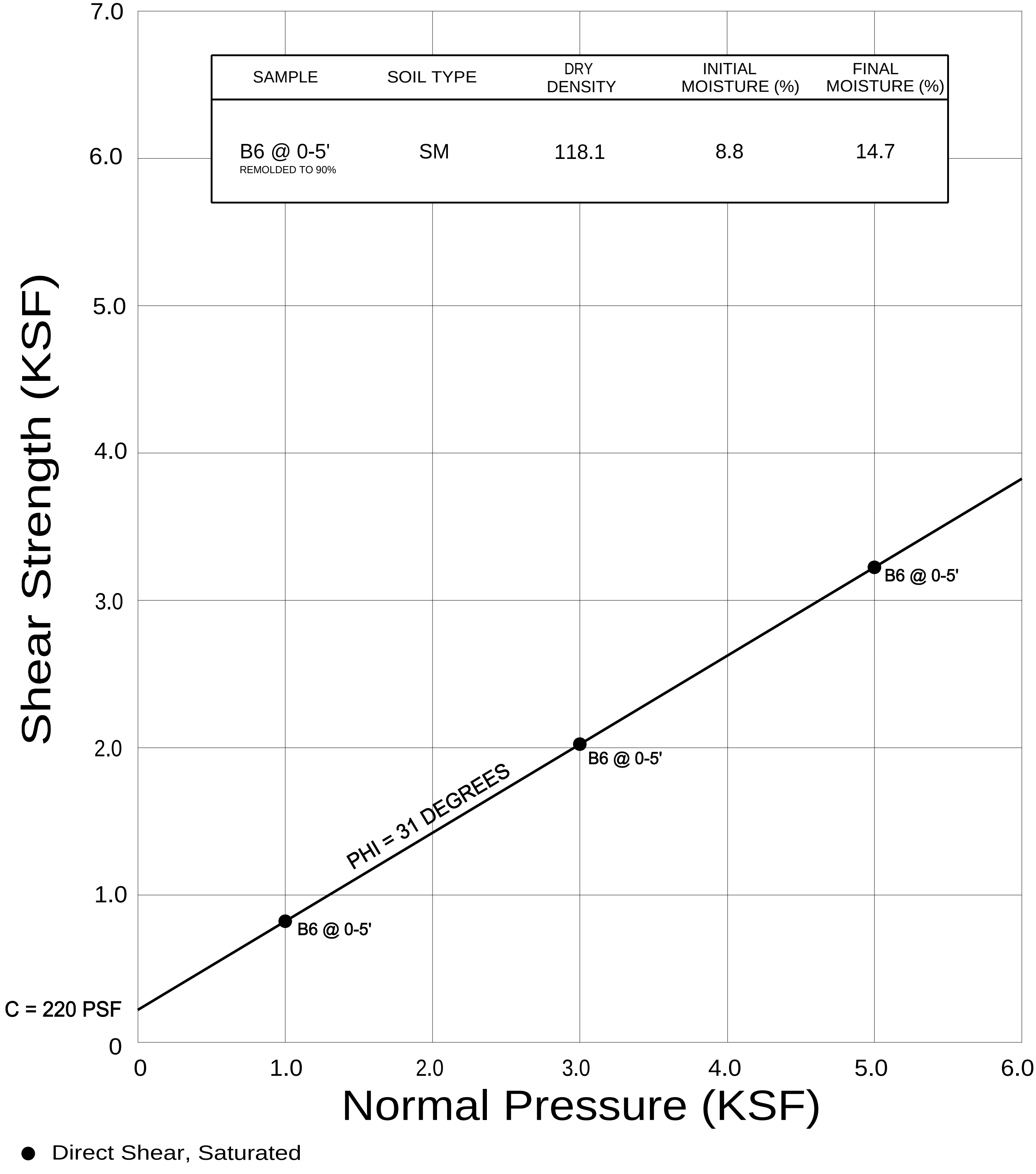
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, grain size distribution, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B9. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6 @ 0-5' REMOLDED TO 90%	SM	118.1	8.8	14.7

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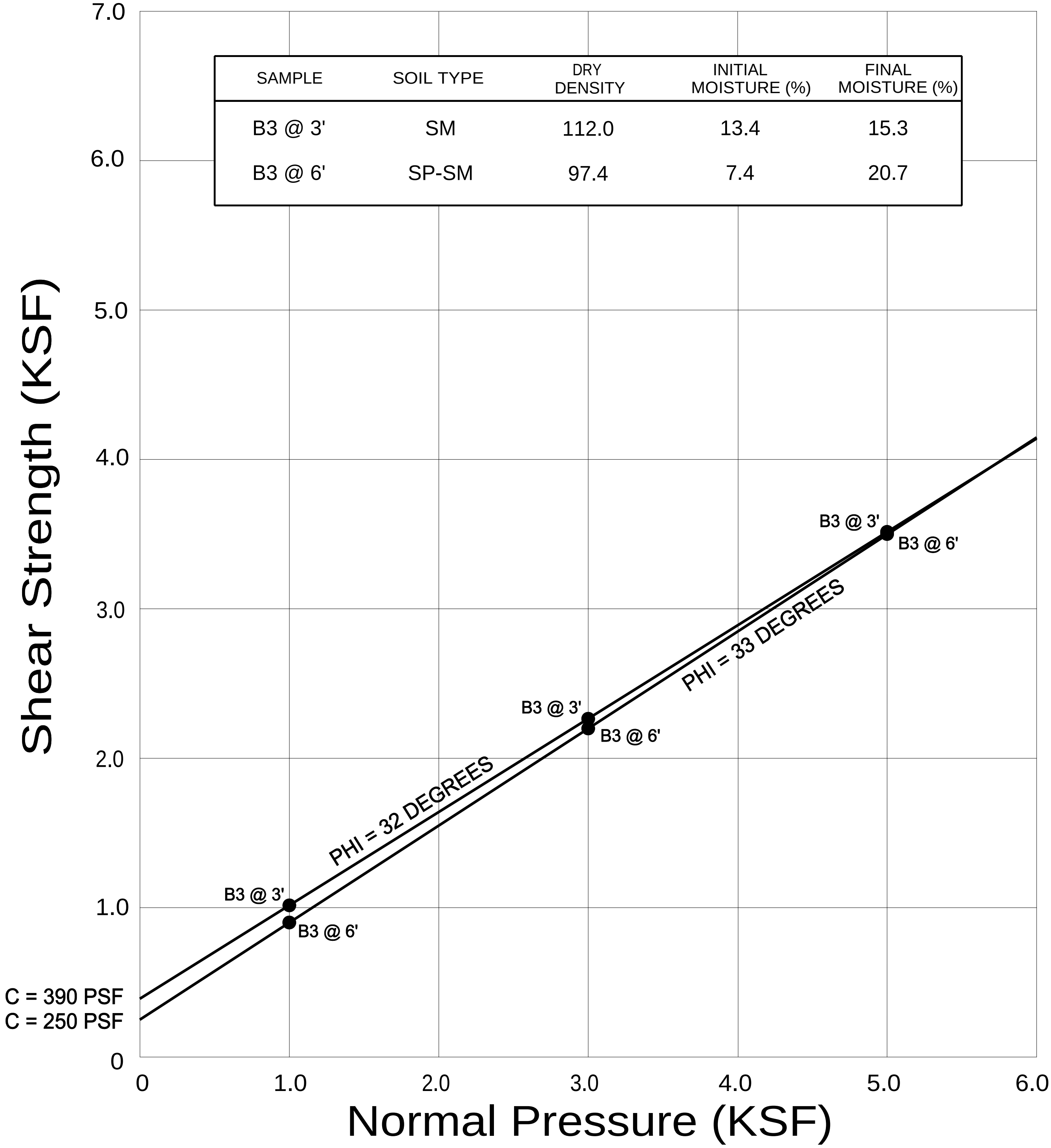
PZ		9000

DIRECT SHEAR TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 3'	SM	112.0	13.4	15.3
B3 @ 6'	SP-SM	97.4	7.4	20.7

● Direct Shear, Saturated

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PZ		9000

DIRECT SHEAR TEST RESULTS

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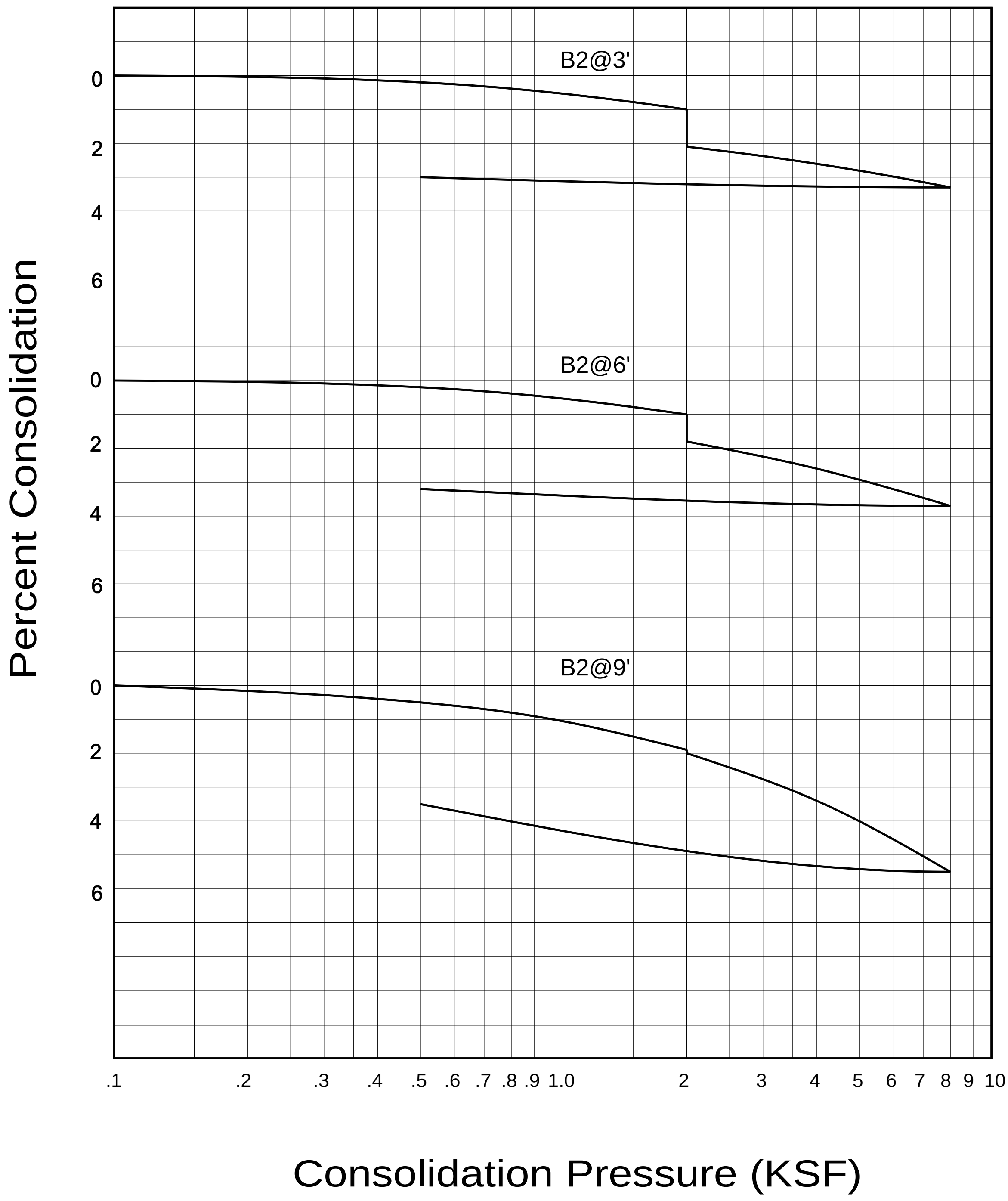
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B2

WATER ADDED AT 2 KSF



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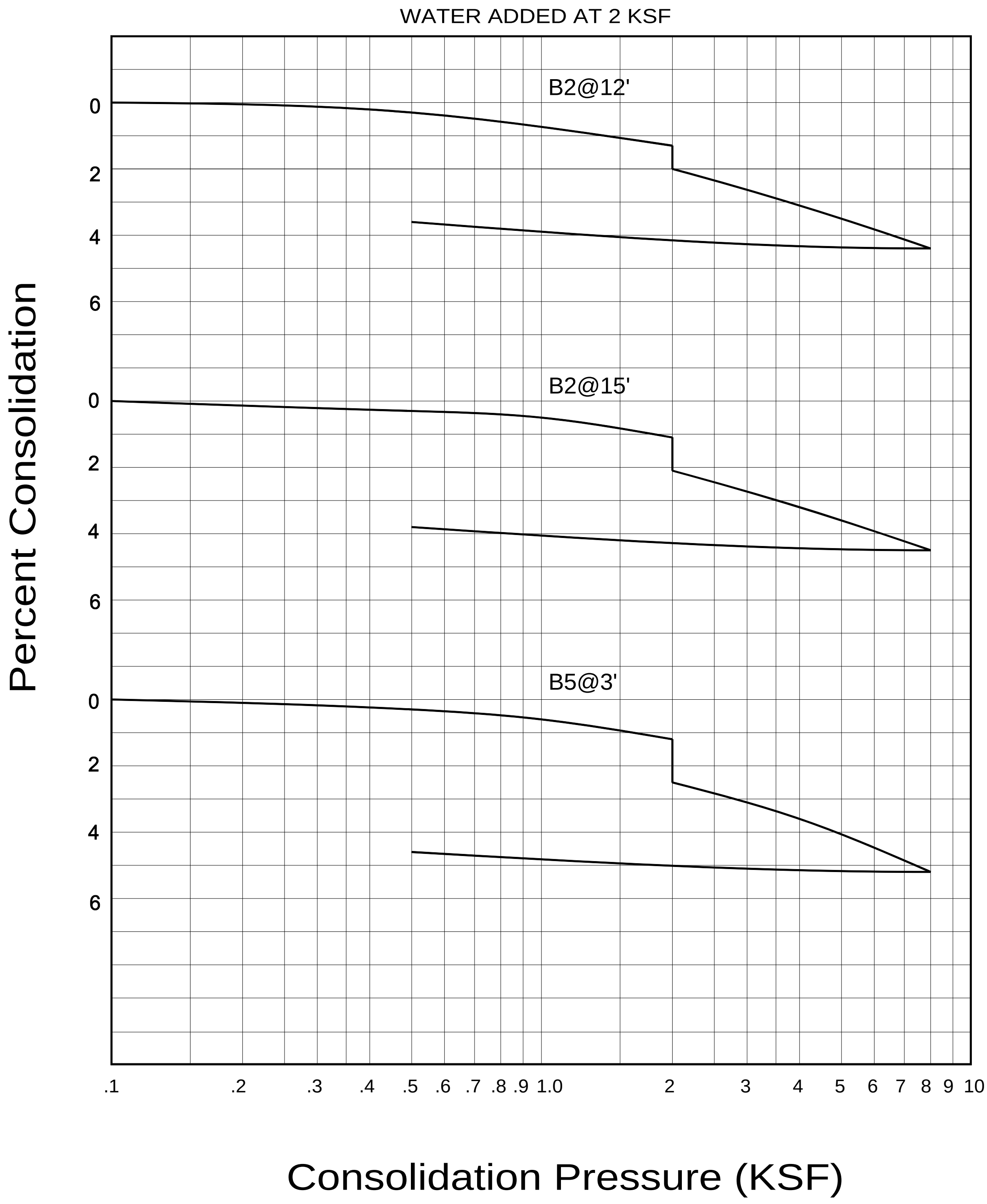
CONSOLIDATION TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL

LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B3



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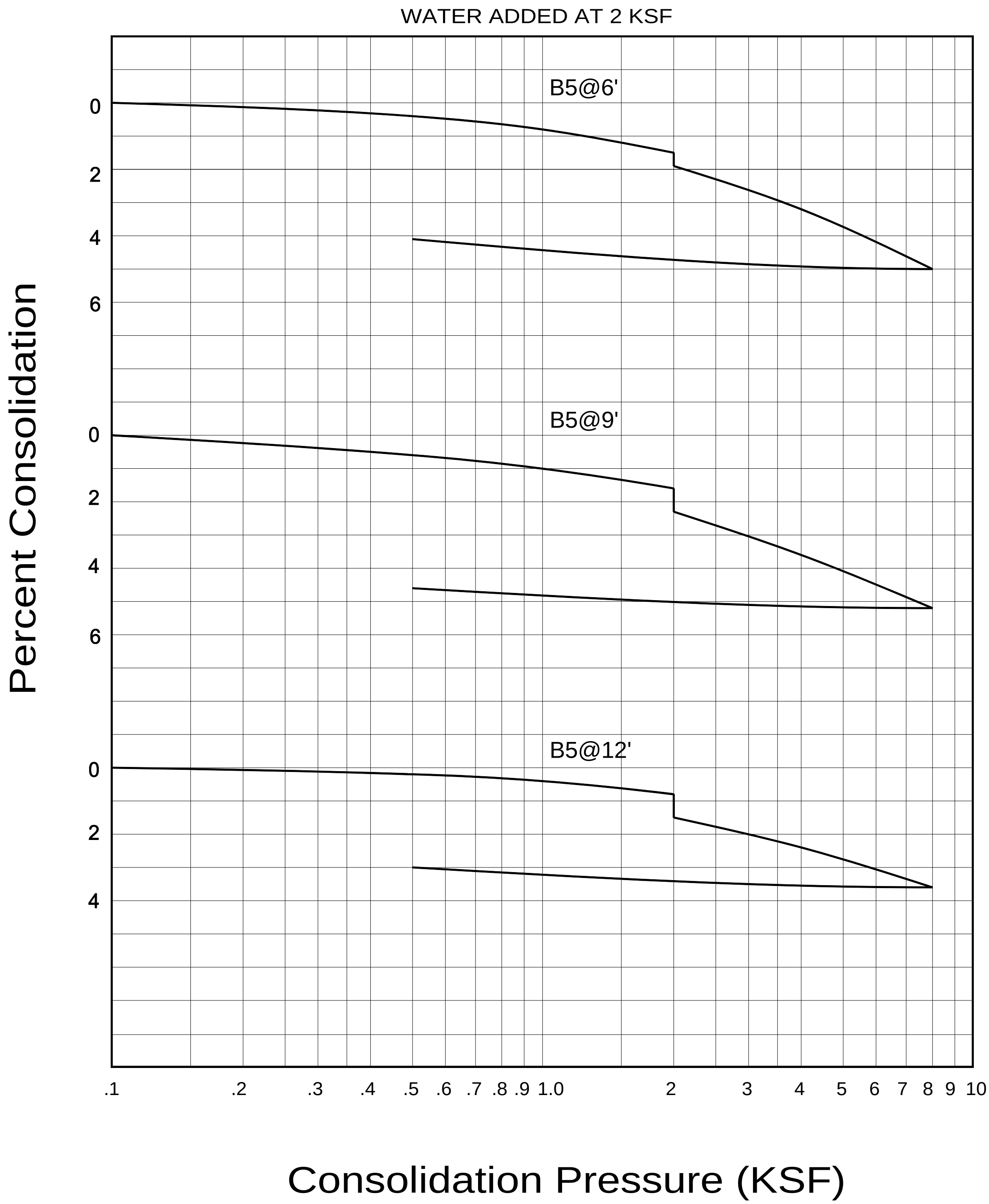
CONSOLIDATION TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL

LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B4



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PZ		9000

CONSOLIDATION TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL

LOS ANGELES UNIFIED SCHOOL DISTRICT

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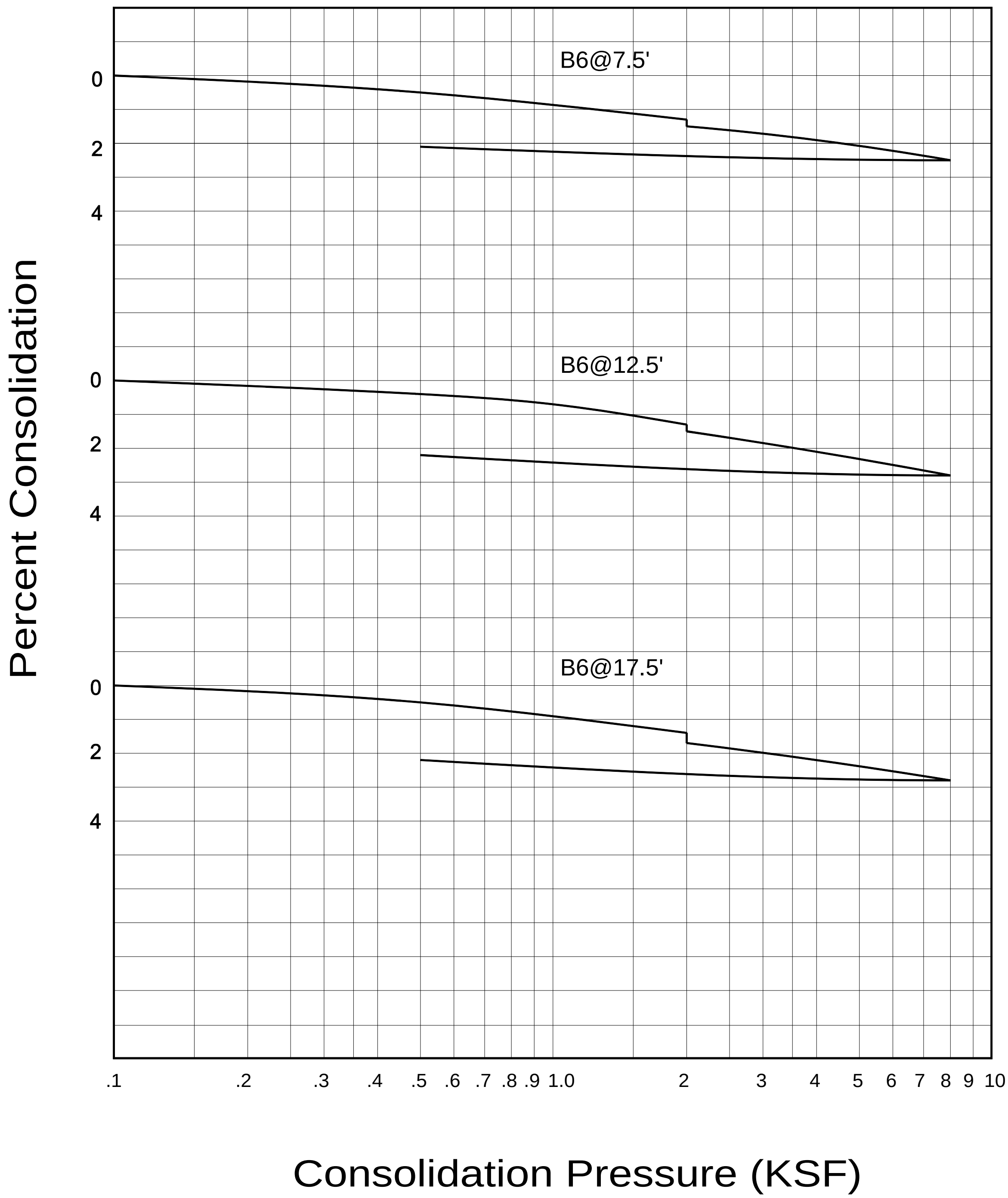
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B5

WATER ADDED AT 2 KSF



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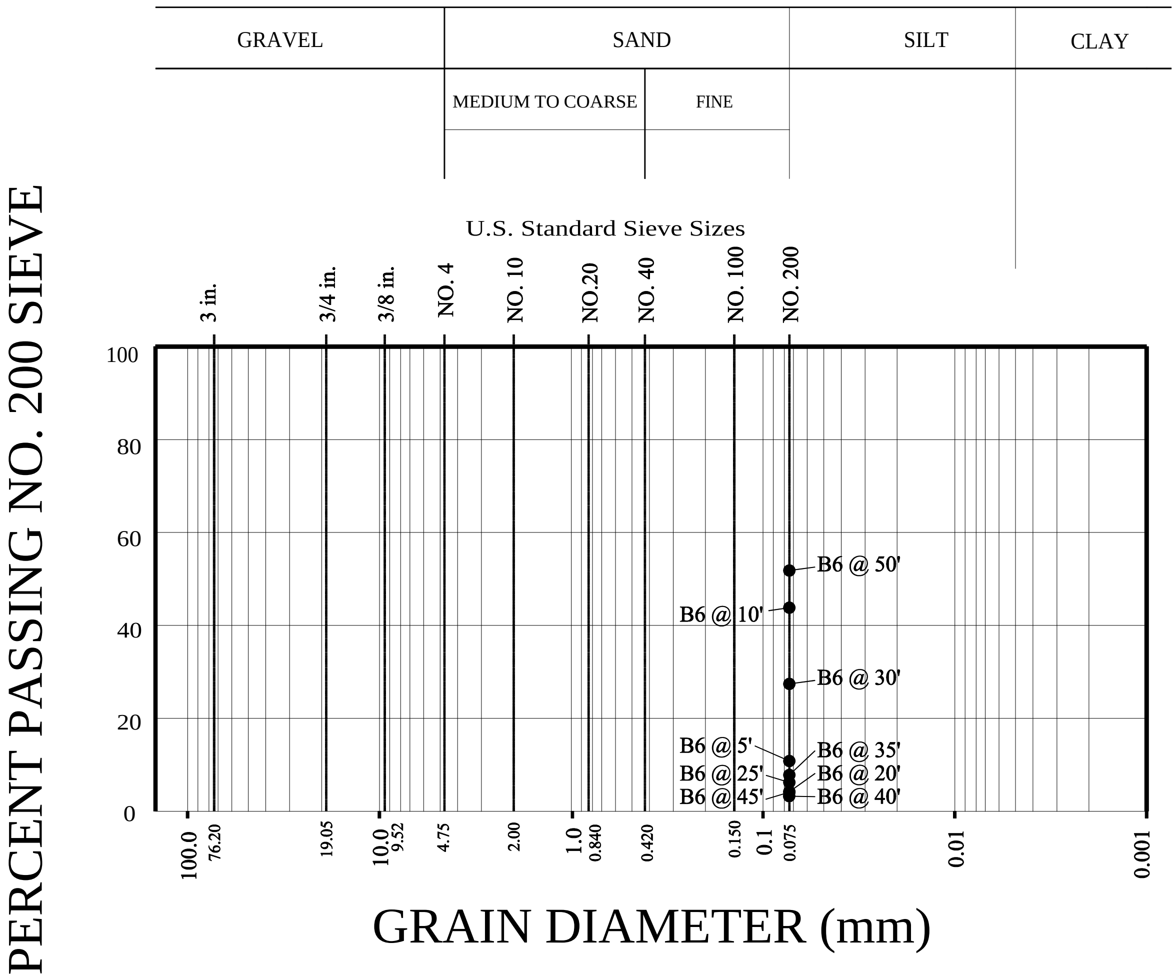
CONSOLIDATION TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B6



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B6 @ 5'	10.8
B6 @ 10'	43.8
B6 @ 20'	4.2
B6 @ 25'	6.2
B6 @ 30'	27.4
B6 @ 35'	7.8
B6 @ 40'	3.2
B6 @ 45'	4.1
B6 @ 50'	51.8

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GRAIN SIZE ANALYSIS

HUNTINGTON PARK HIGH SCHOOL

LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

PZ		9000	JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B7

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B6 @ 0-5'	7.4	13.5	120.5	0	Very Low	Non-Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2013 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B6 @ 0-5'	Dark Grey Silty Sand	134.5	7.0

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PZ		9000

LABORATORY TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015	PROJECT NO. A8326-06-62	FIG. B8

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B6 @ 0-5'	8.69	5700 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B6 @ 0-5'	0.008

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B6 @ 0-5'	0.002	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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PZ | | 9000

CORROSIVITY TEST RESULTS

HUNTINGTON PARK HIGH SCHOOL
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6020 MILES AVENUE
HUNTINGTON PARK, CALIFORNIA

JULY 14, 2015 | PROJECT NO. A8326-06-62 | FIG. B9